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Testing of Two Pin-Connected Truss Bridges

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Abstract:

This report describes load tests on two pin-connected steel truss bridges. It was found that, unlike other bridge types, this type of bridge does not possess reserve strength beyond that determined analytically. The main reason for this is that because of a possible lack of fit, one component of a tension chord comprised of two unconnected components may not fully participate in sustaining loads. The report also contains a procedure in which, by establishing equivalence between the various subconfigurations of the test and design vehicles, posting loads can be readily obtained.

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Testing of Two Pin-Connected Truss Bridges

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1/ THE FINNEY BRIDGE

1.1/ Test Summary

Bridge:	Finney bridge, Township of Charlottenburg, MTC site No. 31-236
Type of bridge:	Steel pin-connected truss bridge with transverse laminated wood decking on steel stringers and floor beams.
Posted weight:	9 t
Span/width:	51.21/4.67 m
Angle of skew:	0°
Date of test:	August 16, 1984
Test Engineer:	Baidar Bakht
Job Captain:	Zdislaw Knobel
Supervisor, Instrumentation:	Bill Kwok
Maximum load applied:	330 kN (34 t) test vehicle brought on to the bridge at crawling speed.
Responses monitored:	Strains in truss members, stringers and floor beams, and deflections of one truss.
Conclusions:	Either post bridge with three level posting of 9, 16, 22 t or, for single level posting, leave the posting sign as it is.

1.2/ Bridge Description

The Finney bridge, two views of which are shown in Figure 1.1a and b, is a one lane structure with pin-connected steel trusses spanning a distance of 51.21 m. The trusses are 6.71 m tall. The top chord is continuous, but the remaining chord members are all pin-connected. Details of the bridge, which was constructed at the turn of the century, are given in Figure 1.2. It can be seen in Figure 1.1b that the tension chord members are composed of unconnected independent components, each with a square or a rectangular cross-section.

The floor systems consist of a 100 mm deep transverse nail laminated wood decking which rests on five steel and two wood stringers. The stringers in turn rest on floor beams which are connected to the truss nodes and have a spacing of 5.69 m. The stringers are continuous over two spans. A cross-section of the floor system is shown in Figure 1.3a. Each floor beam is 381 mm deep and has 136 mm wide flanges. The stringers are 203 mm deep and have 102 mm wide flanges. Both the floor beam and stringer sections do not conform to today's standard sections, nor could these sections be located in older section tables.

An imprint of the exposed end of a floor beam was made on a piece of paper, and the various dimensions measured from the imprint. The moment of inertia of the floor beam section was calculated to be equal to 264×10^6 mm^4.

As shown in Figure 1.3a, the outer stringers are of wood and not in contact with the floor beams; they appeared to be hanging from the wood deck. It did not appear safe to bring the test vehicle on to a floor system in which the two outer stringers were non-functional. Therefore, it was decided to make the wood stringer functional before the test by inserting wooden wedges between the stringers and floor beams. A view of the flooring system from the underside is shown in Figure 1.3b.

Before testing the bridge, the eyebar connections of the web members to the top chords of the truss were carefully inspected. The connection of one of the bars in the south truss was found to have a crack. This connection is identified in Figure 1.2. As shown in Figure 1.1, the bridge is posted for 9 t.

1.3/ Details of Instrumentation

Due to a limited number of channels on the data acquisition system, it was decided to instrument only the south truss. Strains on truss chord members were monitored through demountable strain transducers: the locations of the transducers and deflection transducers on the south truss are shown in Figure 1.4 together with their identifying numbers.

All the steel stringers of the second panel from the west end of the bridge were instrumented by demountable strain gauges installed at mid-span bottom flange. The top flange of one of the stringers was also instrumented by an electrical resistance strain gauge. A demountable strain transducer was installed at the mid-span bottom flange of each of four floor beams. Locations of the strain recording instruments and their identifying numbers are shown in Figure 1.5.

1.4/ Details of Loading

It is usual in a bridge test to first load the structure with the test vehicle carrying a relatively small load. Depending upon the measured response of the bridge, the gross weight of the vehicle is gradually increased until it reaches an optimum level.

For the test on the Finney bridge, the vehicle with a gross weight of 330 kN (≃34 t) was first brought on the bridge. The measured response of the structure, however, indicated that a higher load level will most likely induce a permanent deformation in some components of the structure. It was decided, therefore, not to proceed with the test at higher load levels.

Details of the vehicle, its longitudinal and transverse positions are given in Figure 1.6. The test vehicle was first brought on the bridge at crawling speed, and the instruments were read corresponding to the various longitudinal positions, identified in Figure 1.6, while the

vehicle was in motion. The second time the vehicle was again brought on to the bridge at a crawling speed, but this time it was stopped at the various positions along the bridge for the reading of the instruments. The very small differences in the two sets of readings confirmed that the former method in which the test vehicle does not have to stop on the bridge incurs barely appreciable errors.

1.5/ Observed Behaviour

1.5.1/ Truss Behaviour

All the recorded strain data pertaining to the south truss are listed in Table 1.1.

Compressive strains at various locations in the top chord are plotted in Figure 1.7 against longitudinal vehicle positions. The fact that at a given section the strains in the cover plate are different from those in the farther channel flanges, suggests the presence of substantial bending moments. The strain levels are low, however, corresponding to a maximum stress of about 38 MPa. Clearly, the top chords of the trusses are not the weak link in the load carrying capacity of the bridge.

Strains in the bottom chord due to the various vehicle positions are plotted in Figure 1.8. It can be seen that the two components of the same panel of the bottom chord do not share the force equally, and that their sharing of this load does not depend upon the relative position of the component with respect to the bridge. For example, in the third panel from the west support, the outer component sustains nearly all the bottom chord force. In the next panel, i.e. the fourth one from the west abutment, the situation reverses. In this case the inner component carries most of the force. Yet there are panels such as the first and fifth from the west support, in which the two components share the force more evenly. It is likely that the uneven sharing of forces is caused by a lack of fit at the pin connections.

In spite of the uneven sharing of the force, the bottom chords of the trusses do not give any cause for concern. As can be seen in Table 1.1 and Figure 1.8, the strains in the various components of the bottom

chord are fairly small, being a maximum of 218 x 10^6 mm/mm. This strain corresponds to a tensile stress of about 45 MPa.

Strains in four web members of the truss are plotted in Figure 1.9 against longitudinal vehicle positions. It is obvious from this figure that the two components of these web members share the forces more or less equally. There was one inclined web member, however, in which the force was shared unequally between the two components. In this member, which is identified in Figure 1.4 as member L, strains in the two components are plotted in Figure 1.10. Attention is drawn to the different scales of Figures 1.9 and 1.10.

Web member L was the main reason that a decision was taken to stop the test after the first level of loading. As can be seen in Figure 1.10, the inner component carried most of the member forces. The maximum compressive and tensile stresses in this component due to the vehicle loading were found to be 85 and 243 MPa respectively. In spite of the tensile stress being as high as 243 MPa, all the strains were completely recovered upon the removal of the live load. This suggests the presence of some dead load compressive stresses in the component.

Measured vertical and horizontal deflections of the south truss are given in Table 1.2. It can be seen that there was no horizontal movement at the eastern support, which is designed as fixed. The western support which is designed to permit horizontal movement did undergo horizontal movements of up to 1.35 mm. A less restrained bearing would have permitted much larger movement. One of the expansion bearings is shown in Figure 1.11. It can be appreciated that the expansion bearing, in this condition, cannot be very effective in permitting horizontal movement.

It can be seen in Figure 1.8 that the end panel of the bottom chord is subjected to tensile strains when the vehicle is close to the panel. However, as the vehicle is moved away from this end panel, the strains in its bottom chord begin to reduce until they become compressive when the vehicle is near the other end of the truss. This phenomenon, which is also observed in other truss bridges [5], is due to the bearing restraint against horizontal movement and can be justified as follows.

The Finney bridge truss was analysed for various load cases for two different support conditions. In one condition, both the truss supports were restrained against horizontal movement; and in the other condition, one support was restrained and the other freed. Bottom chord forces in three end panels due to to various loading cases in the two trusses are given in Table 1.3. It can be seen that the same pattern of behaviour as observed can be replicated analytically.

1.5.2/ Floor System Behaviour

All the recorded data pertaining to the floor system are recorded in Table 1.4.

Floor beam strains for the various vehicle positions are plotted in Figure 1.12. The maximum observed strain corresponds to a stress of about 127 MPa. This stress was mainly caused by the 140 kN (14 t) dual axle tandem of the test vehicle.

It was confirmed that the live load strains in the floor beams are in the same proportions as the beam reactions which are obtained by considering that the floor system is continuous over two consecutive panels.

Mid-span strains of stringers in the second panel from the west support were found to be the maximum under load position No. 3. These strains are plotted in Figure 1.13. The maximum bottom flange strain corresponds to a stress of about 108 MPa.

Only one of the stringers, namely No. 6 as identified in Figure 1.13, was installed with strain gauges at both the top and bottom flanges. As shown in Figure 1.13, the top flange strain in this girder was not always the same as the bottom flange. In many cases the magnitude of the top flange strain was larger than that at the bottom, suggesting the presence of restraint forces at the interfaces of the stringer and floor beams. The restraint forces, however, are not large enough to significantly affect the load carrying capacity of the stringers.

2/ THE STEPHENSON TOWNLINE BRIDGE

2.1/ Test Summary

Bridge:	The Stephenson Townline bridge, Town of Bracebridge, MTC site No. 42-62
Type of bridge:	Steel pin-connected truss bridge with transverse laminated wood decking on steel stringers and floor beams
Posted Load:	Triple posting of 9, 14 and 18 t
Spans/width:	40.0, 6.1/5.18 m
Angle of skew:	0°
Date of test:	October 18, 1984
Test Engineer:	Baidar Bakht
Job Captain:	Zdislaw Knobel
Supervisor, Instrumentation:	Bill Kwok
Maximum load applied:	330 kN (34 t) test vehicle brought on the bridge at crawling speed
Responses monitored:	Strains in truss members, stringers and floor beams and deflections of one truss.
Conclusions:	The existing three level posting can be marginally improved to 9, 16, 21 t.

2.2/ Bridge Description

The Stephenson Townline bridge, a view of which is shown in Figure 2.1, is a one-lane structure with pin-connected steel trusses spanning a distance of 40.0 m. The trusses are 6.90 m tall, 5.18 m apart. As in the Finney bridge, the top chord is continuous but the remaining truss members are all pin-connected. The trusses were fabricated in 1892 and relocated to their present location circa 1922. In addition to the main span, the bridge also has an approach span of 16.15 m. Relevant details of the bridge are shown in Figure 2.2.

As shown in Figure 2.3a, the bottom chord of the north truss is buckled in two panels near each end of the truss. The buckled bottom chord does indeed suggest that the floor system, in particular the stringer closest to the truss, share the bottom chord forces. The bottom chord of the south truss was found to be straight, as shown in Figure 2.3b. Locations of the bottom chord buckles are identified on the plan in Figure 2.3c.

A typical cross-section of the floor system in the main truss span is shown in Figure 2.4. As shown in the figure, the floor system consists of a 140 mm deep transverse nail laminated wood decking resting on six W250 x 49 stringers, which are in turn supported by steel I beams reinforced by steel plates attached to the two flanges. The stringers are continuous over two panels. Stringers along a longitudinal line are connected by web splices. Lugs are welded to the top flanges of floor beams to provide four-bolt connections to the stringer. A view of the floor system from the underside is shown in Figure 2.5.

The approach span has the same decking and stringer as the main span; its stringers are also connected to corresponding main span stringers by web splices. Stringers of bolt spans are attached with steel shoe plates at the abutment and pier ends. These shoe plates rest on steel bearing plates. The bearing connections are designed to permit longitudinal movement at the abutments and no movement at the pier.

The bridge is posted with a three level posting of 9, 14 and 18 t as shown in Figure 2.6. The three posting limits apply to single unit trucks, trucks with a semi-trailer or one trailer, and truck trains respectively.

2.3/ Details of Instrumentation

It was decided to instrument the south truss in which the bottom chord is straight. Electrical resistance gauges and strain transducers were attached to a number of locations which are identified in Figure 2.7. Also, as shown in this figure, a number of deflection transducers were attached to the south truss to measure its vertical deflection and the horizontal movement of the supports.

All stringers of the second panel from the pier and four floor beams were instrumented with electrical resistance strain gauges and strain transducers. Locations of the instruments are shown in Figure 2.8.

2.4/ Details of Loading

The bridge was first tested with the empty test vehicle having a gross weight of 218 kN (22 t). When the measured response showed that the bridge could safely sustain the next level of loading, the vehicle with a gross weight of 330 kN (34 t) was brought on the bridge. As in the case of the Finney bridge, the Stephenson Townline bridge was also not subjected to a higher load level.

Details of the vehicle and its axle weights for the two levels of loading are shown in Figure 2.9 which also shows the transverse and longitudinal positions of the vehicle for the various load cases. For the first level of loading, readings were taken twice: firstly when the vehicle travelled at crawling speed on the bridge without stopping, and secondly when the vehicle again travelled at a crawling speed but was made to stop at the specified locations. Again it was confirmed that there was little difference between the two sets of readings. For the second level loading, the former procedure was adopted.

2.5/ Observed Behaviour

2.5.1/ Truss Behaviour

All the recorded strains in the south truss for the two levels of loading are given in Tables 2.1 and 2.2 respectively.

Compressive strains at various locations in the top chord of the south truss are plotted in Figure 2.10 against longitudinal vehicle positions. From the plotted data it can be seen that the horizontal portion of the chord appears to be free from out-of-plane bending. The inclined compression chord, unlike the case of the Finney bridge is, however, subjected to some out-of-plane bending. The relatively low measured strains confirmed that the compression chord has ample reserve strength beyond the applied load level.

Recorded vertical deflections of the south truss and horizontal deflections of its supports are given in Table 2.3 for the various load positions. It can be seen that in spite of having a span shorter by some 22%, the maximum horizontal movement of the Stephenson Townline bridge under the same vehicle is larger by about 100%. This suggests that the bearings of the former bridge are less restrained than those of the latter.

Strains in the bottom chord are plotted in Figures 2.11 and 2.12 against longitudinal positions. The same strains are plotted separately in Figure 2.13 for each of the different vehicle positions along the span. It can be seen that the two components of the chords identified as P and P' in the figures share the forces almost equally. In chords O and N the load sharing between the two components becomes somewhat uneven, and in chord M, which is closest to the support, the inner component takes only a fraction of the total force. It can be noted in Figure 2.12 that the trend of load sharing in the components of this chord, is the same for the two load levels. The same random sharing of the load between the two components of a tension chord as was observed in the Finney bridge, can also be seen in the Stephenson Townline bridge.

An additional surprising feature of this bridge was the behaviour of the bottom chord of the south truss in the end panel near the fixed truss support. As can be seen in Figure 2.13, the maximum strain in one component of this panel was 1433 x 10^{-6} mm/mm, and that in the other it was 118 x 10^{-6} mm/mm. From simple statics it is obvious that the bottom chord forces in the two end panels should be nearly the same. These panels have components of same cross-sections. The fact that the total strains in the two end panels, viz. M and N, are so significantly different suggests that the floor system does not participate with the bottom chord in the end panel.

The above behaviour can be explained as follows. All stringers of the floor system are effectively connected to the floor beams which are themselves connected to the truss nodes in such a way that the bottom chord between two adjacent nodes cannot deform without engaging the stringers. Since there are no floor beams at the truss supports, the bottom chord in the end panels is free to deform without engaging the stringers.

The Finney bridge also does not have floor beams at the truss supports. The strains in an end panel of the bottom chord are small in this bridge, as can be seen in Figure 1.8, not because the floor system shares the bottom chord force, but because its bearings are relatively restrained as discussed earlier.

It is established that the forces in an end panel of the bottom become large when one of the truss bearings is relatively free. In this case the bottom chord forces of both end panels should be affected. This observation is confirmed by the fact that the instrumented end panel was close to the fixed bearing.

Strains in the components of web members of the south truss are plotted in Figure 2.14 and 2.15. Since the two components of all members were not instrumented, it is difficult to conclude if the forces were shared evenly by the two components of each member. In light of the experience with the bottom chord, and with the various tension chords in the Finney bridge, it is prudent to assume that an uneven sharing of forces between

the two components will take place in any one of the components. Even if it is assumed that the tension forces are taken entirely by one component, the maximum resulting stress would correspond to only about 165 MPa.

2.5.2/ Floor System Behaviour

The floor system of the Stephenson Townline bridge was rehabilitated only months before the test. The rehabilitation included the replacement of stringers and strengthening of floor beams by welding plates to their top and bottom flanges. It is not surprising to find that the floor system of this bridge was not a weak link in its load carrying capacity. All the recorded strains in the stringers and floor beams due to the two levels of loading are given in Tables 2.4 and 2.5 respectively. The maximum observed tensile strains in the stringers and floor beams due to level 2 loading correspond to stresses of 50 and 63 MPa respectively.

3/ CALCULATION OF POSTING LOADS

A testing vehicle may not have the same configuration and axle weight ratios as the design or the evaluation vehicle. In order to use the test vehicle as the reference vehicle for evaluating the load carrying capacity, it is necessary to establish some sort of equivalence between the testing vehicle and the three levels of evaluation vehicles as specified in the Ontario Highway Bridge Design Code (OHBDC) [3]. A method is given in Section 3.2 to establish such equivalence.

The concept of the Equivalent Base Length, which is discussed in Reference 1 in detail, is used as the basis of the procedure. For the sake of completeness, it is described briefly as follows.

It has been shown in the above cited reference, that N number of discrete loads can be represented by a uniformly distributed load of total weight W and an Equivalent Base Length B_m, so that the moment and shear envelopes due to the latter load on any beam are nearly the same as those due to the former set of loads. W is equal to the sum of discrete loads, and B_m is given by:

$$B_m = \frac{4}{W} \sum_{i=1}^{N} \left| P_i x_i \right| - \frac{2(N-1)}{bNW^2} \sum_{i=1}^{N} (P_i x_i)^2 \qquad (3.1)$$

where the notation is as shown in Figure 3.1. The load closest to the centre of gravity of the group of loads is taken as the reference load and distances of other axles, x, are measured from this load.

3.1/ OHBDC Evaluation Loads

The three levels of OHBDC evaluation loads, which are three different subconfigurations of the OHBDC design truck provides an upper bound of weights in the W-B_m space. This level, which is referred to as the Maximum Observed Load (MOL) level, is shown in Figure 3.2. The equation for the MOL curve is:

$$W_m = 9.806 \, (20.0 + 3.0 \, B_m - 0.0325 \, B_m^2) \qquad (3.2a)$$

where Wm and Bm are in kilonewtons and metres respectively.

The various relevant subconfigurations of the OHBDC evaluation trucks are also plotted in the W-B_m space in Figure 3.2. Ideally, these points should have fallen on the MOL curve. In practice, the evaluation truck weights are always above the MOL line, with the difference being as large as about 9%. As shown in Figure 3.2, the various relevant subconfigurations for the evaluation loading can be closely represented by:

$$W = 9.806\ (20 + 4.4079\ B_m - 0.1670\ B_m^2 + 0.0036\ B_m^3) \quad (3.2b)$$

where W and B_m are again in kN and m respectively. It should be noted that the above curve is valid only up to a B_m of about 20 m.

3.2/ Equivalence Between Evaluation and Test Trucks

Before seeking equivalence between the evaluation truck and test vehicle, it is necessary to identify those subconfigurations of the test vehicle which have a direct influence on the load carrying capacity of the various components of the bridge.

From calculations it was found and from test results confirmed that:

(a) the load carrying capacities of the floor system and some tension web chords are governed by the rear dual axle group of the test vehicle;
(b) the capacities of other critical truss members are governed by the full test vehicle.

Therefore, for the two bridges under consideration, equivalence with the evaluation trucks should be sought for only two subconfiguration of the test vehicle.

The B_m for the rear dual axle group is 3.20 m and that for the whole vehicle 14.77 m. From Equation 3.2b, the equivalent OHBDC trucks are found to be 318 and 591 kN respectively. It can be seen that the total staticly applied load is equivalent to 50% of the evaluation load, the governing load for the floor system is equivalent to only 46% of the corresponding evaluation load.

3.3/ Calculation Procedure

The design equation for any structural component according to the limit states design method is:

$$\phi R_n \geq D + L \tag{3.3}$$

where ϕR_n is the factored nominal resistance of the component; D is the dead load effect due to factored loads; and L is the load effects due to factored live loads which include the Dynamic Load Allowance (DLA).

Consider a bridge which has safely sustained a proof load W_t, the load effects due to which are denoted by L_t. It can be demonstrated that for this bridge:

$$L_t \leq \phi R_n - D \tag{3.4}$$

The corresponding load effects due to a load $\alpha_L W(1 + I)$ are denoted by L_e, where α_L is the live load factor, W is the Evaluation Truck weight and I is the DLA.

To quantify the load carrying capacity of substandard bridges, the OHBDC specifies a scale down factor, F, which is obtained from the following equation:

$$F = (\phi R - D)/L_e \tag{3.5}$$

By using Equation 3.4, the safe upper bound value of F is given by:

$$F = L_t/L_e \tag{3.6}$$

when the load effects are linearly related to load, Equation 3.6 can be rewritten as:

$$F = \frac{W_t}{\alpha_L W (1 + I)} \tag{3.7}$$

It is noted that W is obtained from Equation 3.2 corresponding to the base length of the relevant subconfiguration of the testing vehicle. Values of α_L and I can be obtained from the OHBDC [3].

Once the values of the scale down factors, F, are determined for the relevant subconfigurations of the testing vehicle, the posting loads corresponding to the three levels of evaluation loads can be readily obtained from the posting load charts of the OHBDC. It is noted that the charts of the first edition of the OHBDC [2] provided conservative estimates of the posting loads for the upper ranges of F. When this unnecessary conservativeness was eliminated the charts of the second edition [3] were inadvertently made over-conservative in the lower ranges of F. It is proposed that until these charts are further amended, the higher levels of the posting loads as obtained from the two sets of charts, be used for posting. Any possible errors involved in this process are easily compensated by the following two factors.

(a) The load factor α_L, besides other uncertainties is also supposed to account for possible errors in analysis. In a bridge test the possibility of error in analysis is obviously eliminated. Therefore, the actual live load factor should be somewhat smaller. The code specified higher value of α_L used in Equation 3.7 should lead to a conservative, i.e., safer, estimate of F.

(b) The proof test load is obviously smaller than the failure laod. The difference between the two loads which clearly depends upon the type of structure and its details, should more than compensate for an unlikely error incurred by the above procedure.

3.4/ Posting Loads for the Finney Bridge

The first flexural frequency of the Finney bridge truss is found to be 8.3 Hz. The DLA corresponding to this frequency is 0.25 according to the OHBDC [3]. Since the roads immediately on either side of the bridge are about at right angles to the bridge axis, the trucks are not likely to travel on the bridge at speeds of greater than 25 km/h. Accordingly, the above DLA for truss members can be reduced by 50% to give a net DLA of 0.125. Similarly, for floor beams the DLA is taken as half of 0.40,

being equal to 0.20. Other relevant data used for evaluation are as follows:

Performance factor, ϕ, for steel in flexure	= 0.95
Yield stress of steel in flexure	= 230 MPa
Maximum flexural stress in a floor beam due to factored dead load	= 5 MPa
Live load factor	= 1.25

The two weak links in the load carrying capacity of the bridge are the tension chords of the truss, and the floor beams. The capacities of the former are governed by the full testing vehicle. Because of the very high recorded strains, it is assumed that the 330 kN testing vehicle constitutes the limiting live load for the trusses. This limiting live load is equivalent to the factored live loads including the DLA. It is noted that the above assumption is a safe side one because all the recorded strains were completely recovered upon the removal of the load indicating that the structure was still elastic and that the failure load would be greater than the 330 kN testing vehicle.

By simple calculations it can be shown that the rear dual axle of the test vehicle would have to carry a total weight of 245 kN to load a floor beam to a maximum stress level of 218.5 MPa, which is the factored nominal stress. In the calculation of the posting loads, this dual axle weight is used for floor beams, and as discussed above, the full vehicle for the truss chords.

The various steps of calculation are given in Table 3.1. It can be seen that the bridge should either be posted for a one-level posting of 9 t, or a three-level posting of 9, 16 and 21 t.

3.5/ Posting Loads for the Stephenson Townline Bridge

Unlike the case of the Finney bridge, the floor system of the Stephenson Townline bridge is not the weak link in the load carrying capacity of the bridge. The capacity of the latter bridge is governed by the bottom chords, and possibly other tension chords, of its trusses. As for the Finney bridge, the 330 kN test vehicle can again be considered as the

limiting vehicle that the bridge can sustain. In this case, the calculations of the posting loads corresponding to the full test vehicle for the Finney bridge would also apply to the Stephenson Townline bridge. As can be seen in the second row of Table 3.1, these calculations lead to a one-level posting of 9 t, or a three level posting of 9, 16 and 21 t.

4/ CONCLUSIONS

It has been shown that unlike other bridge types, a bridge with pin-connected trusses may not have reserve strengths beyond those calculated analytically. The main reason for this observation is that, because of a possible lack of fit, one component of a tension chord composed of two unconnected components may not fully participate in sustaining loads.

The posting loads for the two bridges are recommended to be as follows:

The Finney Bridge: One-level posting - 9 t; or
three-level posting - 9, 16, 21 t.

The Stephenson Townline Bridge:
One-level posting - 9 t;
three-level posting - 9, 16, 21 t.

5/ REFERENCES

[1] Csagoly, P.F. and Dorton, R.A., "Proposed Ontario Bridge Design Load," RR 186, Ministry of Transportation and Communications, Downsview (MTC), Ontario, 1973.

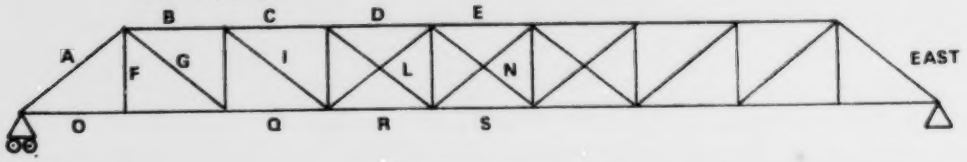
[2] Ontario Highway Bridge Design Code, MTC, Downsview, Ontario, 1st Edition, 1979.

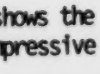
[3] Ontario Highway Bridge Design Code, MTC, Downsview, Ontario, 2nd Edition, 1983.

[4] Sanders, W.W., Elleby, H.A., and Kleiber, F.W., "Ultimate Load Behaviour of Full Scale Highway Truss Bridges," Summary report to Iowa State Highway Commission, Engineering Research Institute, Iowa State University, Ames, Iowa, September, 1975.

[5] Tharmabala, T., "Structural Evaluation and Testing of Mattawa River Bridge," SRR-84-02, MTC, 1984.

Table 1.1/ Recorded Strains in the South Truss of the Finney Bridge, mm x 10^6/mm



	member	cross-section	gauge no.	strain x 10^6 due to load position no.'s (see Fig.6)							
				1	2	3	4	5	6	7	8
TOP CHORD	A		1	-74	-151	-184	-184	-151	-137	-111	-80
			2	-50	-77	-69	45	-33	-19	-8	0
			3	-77	-140	-169	-164	-150	-130	-102	-72
	B		8	-41	-92	-123	-115	-92	-68	-45	-23
			51	-38	-89	-123	-120	-95	-74	-51	-28
	C		13	-36	-79	-128	-153	-141	-111	-72	-41
			14	-35	-77	-127	-153	-139	-111	-78	-46
	D		53	-23	-55	-90	-114	-125	-112	-81	-44
			20	-34	-82	-141	-179	-175	-142	-98	-57
			21	-28	-69	-106	-132	-145	-130	-91	-51
	E		26	-25	-62	-104	-127	-128	-123	-96	-55
			27	-27	-68	-117	-144	-146	-144	-119	-72
WEB	F		6	158	157	68	-2	3	1	5	3
			7	198	192	92	-8	-6	-2	-5	-5
	G		11	6	149	276	300	248	194	137	78
			12	12	109	212	233	188	150	105	62
	I		15	-43	-17	161	320	326	245	170	102
			16	-40	-20	118	234	237	180	129	81
	L		22	-20	-22	-5	-5	193	240	153	81
			23	-202	-410	-432	-432	1012	1177	760	455
	N		28	67	148	295	290	-2	-154	-164	-118
			29	55	127	248	232	-10	-159	-127	-76
BOTTOM CHORD	O		4	57	86	65	25	0	-28	-45	-33
			5	96	139	118	61	20	-21	-36	-28
	Q		17	61	134	172	151	111	69	30	15
			18	0	0	0	0	-1	-1	-1	0
	R		24	0	1	0	0	2	1	0	0
			25	19	76	156	190	174	124	78	44
	S		30	9	30	63	107	142	131	96	67
			31	14	55	114	178	218	195	127	77

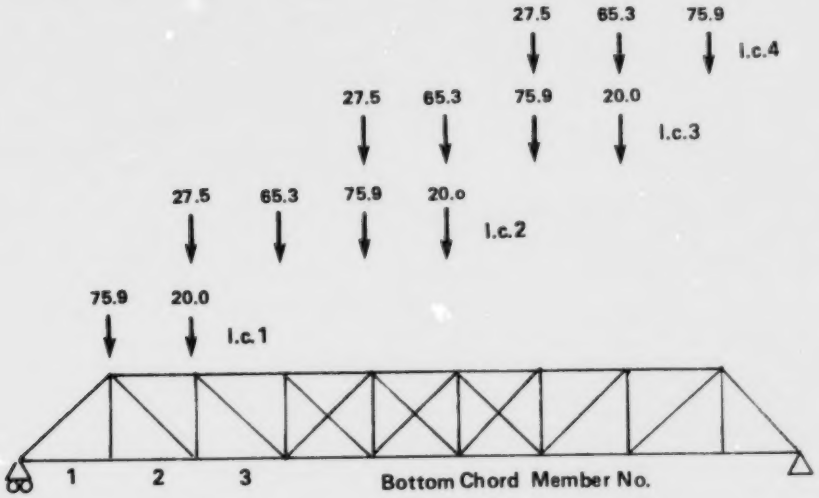
Notes: 1. * shows the component on the outer side of the truss
2. compressive strain shown as -ve

Table 1.2/ Recorded Deflections of South Truss of the Finney Bridge

	Transducer No. (see Fig. 1.4)	Deflection in mm due to load position no. (see Fig. 1.6)							
		1	2	3	4	5	6	7	8
vertical deflection *	46	3.53	8.03	11.53	12.12	10.36	8.13	5.31	2.97
	47	3.48	9.27	16.23	19.79	18.31	14.35	9.88	5.99
	48	3.00	7.85	15.19	21.21	22.38	19.25	13.44	7.95
	49	1.52	4.14	8.35	12.85	16.97	19.02	16.23	10.13
horizontal deflection of supports **	45	0.13	0.76	1.17	1.24	1.32	1.35	1.32	1.19
	50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

* +ve downward
** +ve outward

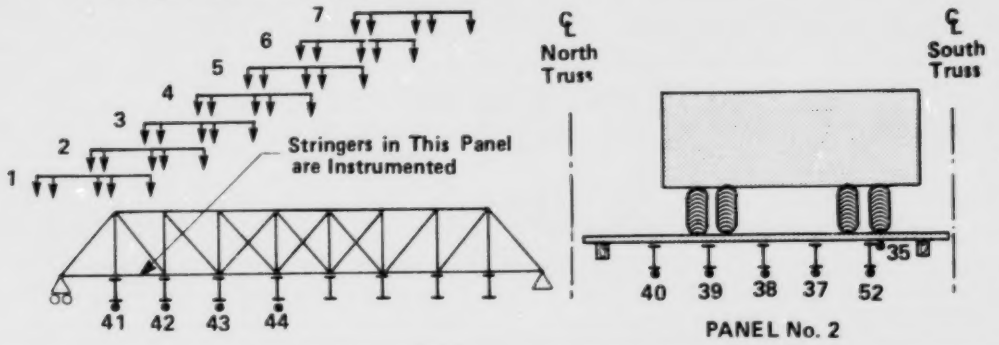
Table 1.3/ Analytical Forces in the Bottom Chord of a Truss



Load Case No.	Force in bottom chord number					
	1		2		3	
	both ends fixed	one end free	both ends fixed	one end free	both ends fixed	one end free
1	25	70	25	70	31	76
2	-38	98	-38	98	60	196
3	-74	62	-74	62	-11	125
4	-64	27	-64	27	-37	54

Note : All Quantities are in Compatible Units.

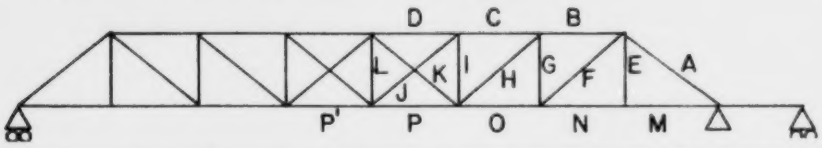
Table 1.4/ Recorded Strains in the Stringers and Floor Beams of the Finney Bridge, mm x 10^6/mm



	gauge no.	strain x 10^6 due to load position no.						
		1	2	3	4	5	6	7
FLOOR BEAMS	41	616	581	244	16	32	37	39
	42	107	444	354	129	-62	-35	-37
	43	4	156	566	571	274	-34	-28
	44	1	0	131	437	358	141	-44
STRINGERS	40	66	106	362	-9	0	-2	-2
	39	50	58	393	-18	-4	-6	-7
	38	44	36	398	-21	1	-1	-2
	37	37	51	511	0	11	9	7
	52	55	46	482	-7	2	0	-2
	35	-55	-76	-524	-8	-19	-18	-2

compressive strain shown as -ve

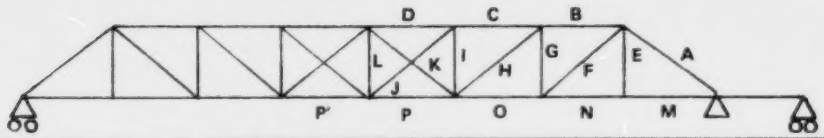
Table 2.1/ Strains in South Truss of the Stephenson Townline Bridge Due to Test Vehicle, Level 1, mm x 10^6/mm

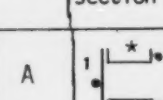
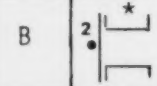
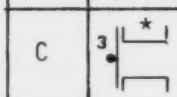
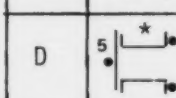
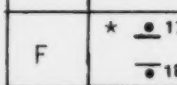
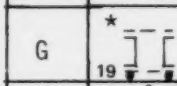
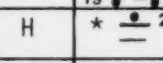
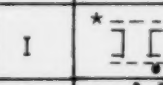
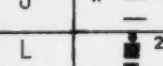
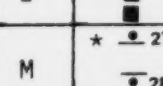
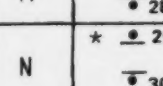
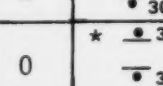
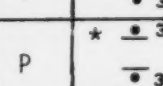
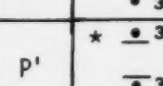


	member	cross-section	gauge no.	strain x 10^6 due to load position no.'s (see Fig.6)							
				1	2	3	4	5	6	7	8
TOP CHORD	A	1 * 12 / 13	12	-77	-134	-136	-132	-108	-86	-57	-33
			1	-43	-65	-62	-55	-40	-27	-15	-5
			13	-58	-98	-99	-95	-78	-62	-43	-21
	B	2 *	2	-54	-109	-113	-113	-90	-67	-44	-19
	C	3 *	3	-35	-95	-107	-113	-104	-80	-50	-23
	D	5 * 14 / 15	14	-28	-72	-84	-93	-98	-90	-62	-27
			5	-34	-102	-126	-141	-144	-125	-78	-36
			15	-25	-66	-77	-88	-92	-86	-59	-26
WEB	E	16	16	247	191	120	25	-2	0	-2	-2
	F	* 17 / 18	17	15	126	155	185	180	149	103	56
			18	47	193	212	240	202	155	103	54
	G	* 19 20	19	62	47	10	-6	-81	-80	-48	-22
			20	55	41	2	-9	-80	-85	-52	-28
	H	* 21	21	-24	50	120	166	238	240	175	106
	I	* 22 23	22	0	0	0	0	0	0	0	0
			23	10	50	41	29	6	-37	-29	-15
	J	* 25	25	-43	-147	-88	-10	126	250	193	104
	L	24	24	45	156	102	39	-97	-219	-174	-90
BOTTOM CHORD	M	* 27 / 28	27	981	790	517	146	11	40	30	25
			28	63	80	70	51	30	20	11	12
	N	* 29 / 30	29	24	32	22	15	6	8	5	2
			30	33	47	38	32	21	19	11	13
	O	* 31 / 32	31	59	161	157	160	126	78	41	28
			32	39	98	91	100	69	43	26	17
	P	* 33 / 34	33	17	81	96	111	106	98	60	38
			34	20	85	99	118	116	99	62	36
	P'	* 35 / 36	35	5	24	40	68	92	102	85	56
			36	4	25	36	57	76	86	64	35

Notes: 1. * shows the component on the outer side of the truss
2. compressive strain shown as -ve

Table 2.2/ Strains in South Truss of the Stephenson Townline Bridge Due to Test Vehicle, Level 2, mm x 10^6/mm



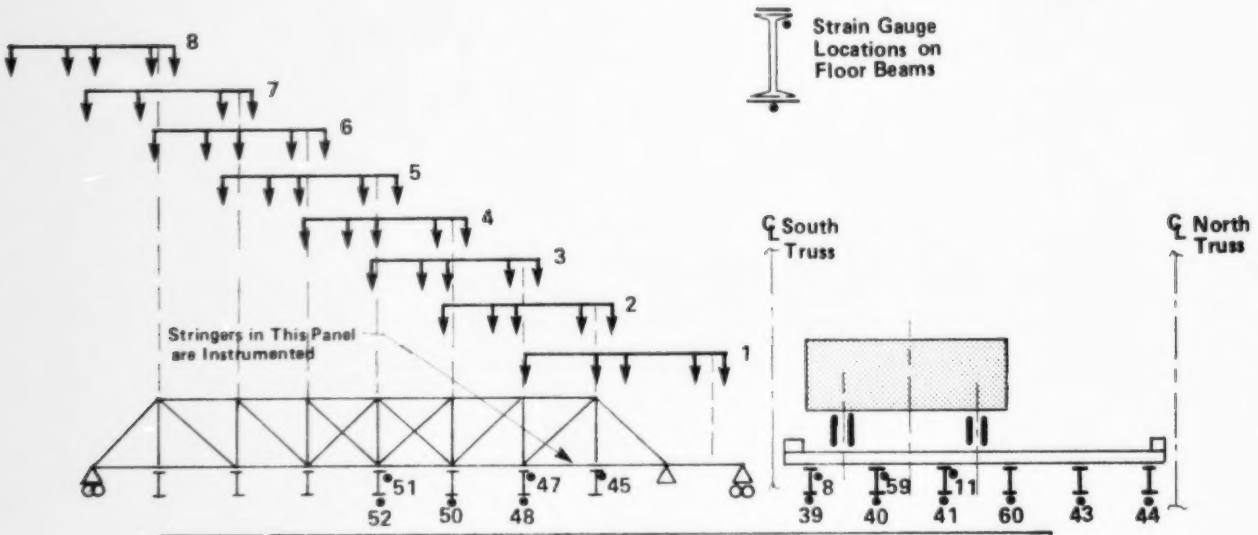
member	cross-section	gauge no.	strain x 10^6 due to load position no. (see Fig. 2.9)							
			1	2	3	4	5	6	7	8
A		12	-100	-199	-202	-191	-167	-127	-95	-52
		1	-58	-100	-99	-84	-66	-46	-28	-11
		13	-73	-139	-143	-135	-113	-88	-62	-32
B		2	-67	-154	-166	-167	-140	-102	-69	-34
C		3	-38	-128	-150	-160	-161	-119	-80	-38
D		14	-33	-97	-114	-130	-142	-137	-101	-47
		5	-39	-139	-172	-192	-217	-192	-128	-61
		15	-30	-91	-107	-122	-135	-132	-100	-47
E		16	365	355	244	63	-5	2	2	0
F		17	12	170	221	285	282	229	152	80
		18	44	246	295	359	308	227	155	83
G		19	70	78	39	25	-121	-125	-83	-41
		20	64	71	29	17	-119	-130	-91	-48
H		21	-23	54	155	225	382	376	268	161
I		23	16	63	70	54	17	-45	-41	-26
J		25	-63	-265	-142	-40	151	387	309	169
L		24	63	273	207	98	-85	-251	-215	-144
M		27	1433	1412	1012	277	16	50	39	28
		28	79	118	111	75	37	14	0	-6
N		29	22	39	33	19	0	2	6	2
		30	35	58	58	45	25	7	-2	-1
O		31	69	220	237	244	198	117	62	34
		32	45	133	140	145	109	61	32	15
P		33	14	96	124	147	161	132	84	46
		34	17	103	137	164	177	143	91	47
P'		35	5	28	47	76	123	140	121	69
		36	1	31	50	74	112	126	101	54

Notes: 1. * shows the component on the outer side of the truss
2. compressive strain shown as -ve

Table 2.3/ Recorded Deflections of South Truss of the Stephenson Bridge Due to Load Level No. 2 (see Figure 2.9)

Transducer No. see Figure 2.7	Deflection in mm due to load position no. see Figure 2.9							
	1	2	3	4	5	6	7	8
53	3.51	9.55	10.19	10.95	10.11	7.19	4.50	2.31
54	3.10	11.51	13.69	15.44	16.15	12.22	7.47	3.91
55	2.67	9.83	12.98	15.95	18.92	17.53	11.58	5.38
56	0.97	3.91	5.26	7.06	10.08	11.81	11.20	7.44
57	0.08	0.05	0.02	0.05	0.00	0.15	0.03	0.05
58	0.89	1.80	1.82	2.26	2.61	2.61	2.72	2.34

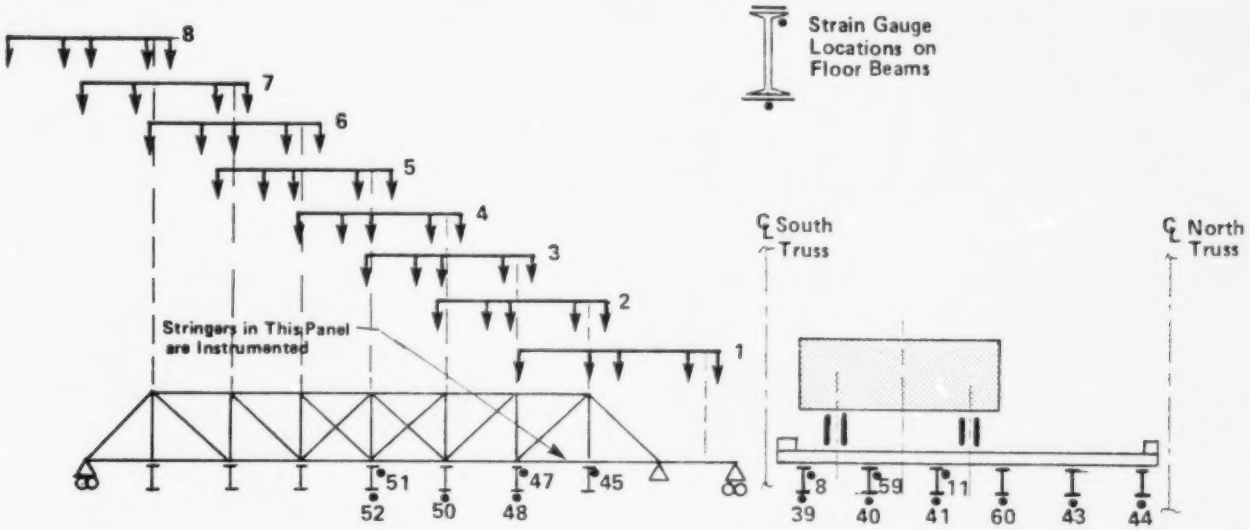
Table 2.4/ Strains in the Floor System of Stephenson Townline Bridge Due to Load Level 1



	Gauge No.	strain x 10^6 due to load position no.							
		1	2	3	4	5	6	7	8
FLOOR BEAMS	45	-163	-134	-80	-23	2	-2	-2	-2
	47	-78	-137	-108	-114	-12	9	5	4
	48	100	183	145	156	30	-4	0	0
	50	2	160	204	197	173	39	6	4
	51	1	-26	-71	-99	-101	-27	-15	9
	52	3	32	99	138	158	-80	-50	-5
STRINGERS	8	-41	-19	-61	-20	8	2	-1	-3
	39	61	33	91	41	-4	4	0	3
	59	-33	-2	-94	-19	12	7	3	1
	40	39	12	76	37	1	8	7	9
	11	-15	3	-89	-20	13	5	5	2
	41	42	-1	131	32	-18	-7	-10	-6
	60	33	12	60	3	-18	-11	-15	-12
	43	51	23	82	22	-6	0	-1	0
	44	40	33	36	14	-6	-8	-12	-10

compressive strains are shown as -ve

Table 2.5/ Strains in the Floor System of Stephenson Townline Bridge Due to Load Level 2

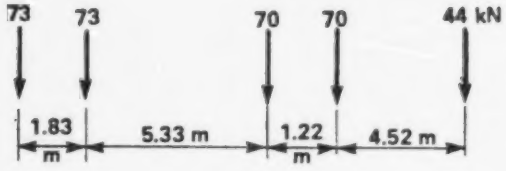


	Gauge No.	strain x 10^6 due to load position no.							
		1	2	3	4	5	6	7	8
FLOOR BEAMS	45	-241	-226	-143	-30	14	9	4	4
	47	-89	-197	-175	-199	-18	28	16	15
	48	116	280	252	290	56	-6	-5	-4
	50	-4	214	307	311	321	76	4	4
	51	2	-18	-67	-113	-157	-190	-8	22
	52	0	33	110	175	248	288	39	-9
STRINGERS	8	-44	-36	-113	-33	16	8	4	0
	39	72	70	164	75	1	12	4	3
	59	-25	-17	-174	-46	21	8	4	0
	40	13	5	151	58	-16	-7	-12	-13
	11	-12	-6	-153	-25	20	7	5	2
	41	38	21	243	57	-24	-4	-10	-11
	60	33	40	171	42	-27	-8	-21	-20
	43	53	38	163	48	-12	-3	-9	-9
	44	59	73	89	41	-1	-1	-14	-12

compressive strains are shown as -ve

Table 3.1/ Calculation of Posting Loads for the Finney Bridge

Dual axle that can be safely sustained by the bridge



full test vehicle

	B_m (m)	total wt., $\bar{W}$ (t) kN	DLA	Eq. static wt., $\bar{W}_s = \frac{\bar{W}}{1 + DLA}$	Eq. OHBDC truck load, w (kN)	F $= \frac{\bar{W}_s}{1.25\ w}$	Posting load (t)		
							lev 1	lev 2	lev 3
Dual axle	3.20	245	0.20	204	318	0.51	9	16	21
Full vehicle	4.77	330	0.125	293	591	0.40	9	16	21

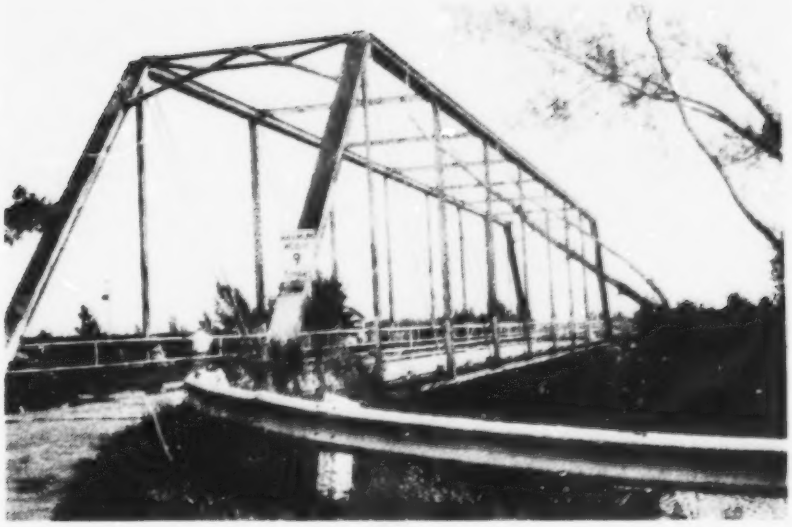


Figure 1.1(a)/A View of the Finney Bridge

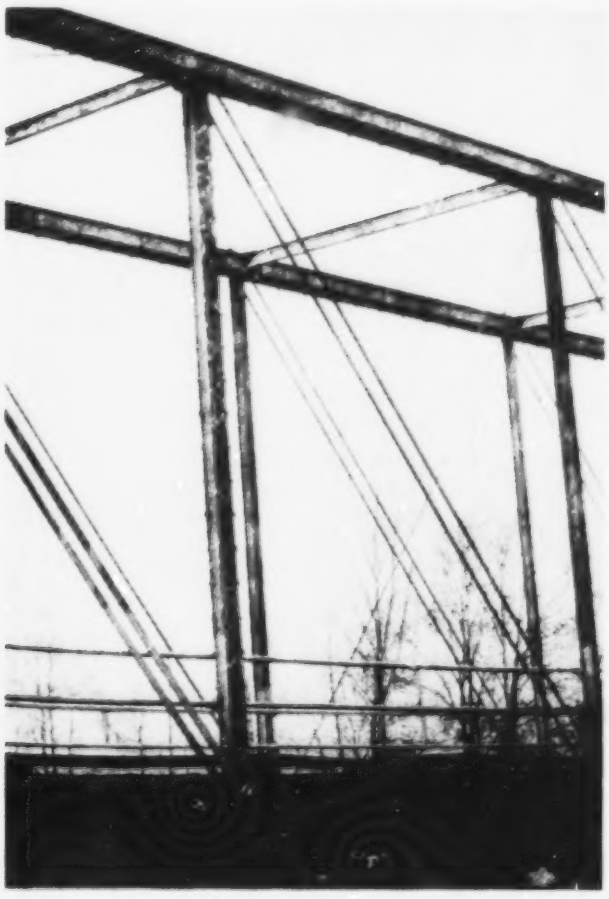


Figure 1.1(b) Another View of the Finney Bridge Showing in Particular some Tension Chords

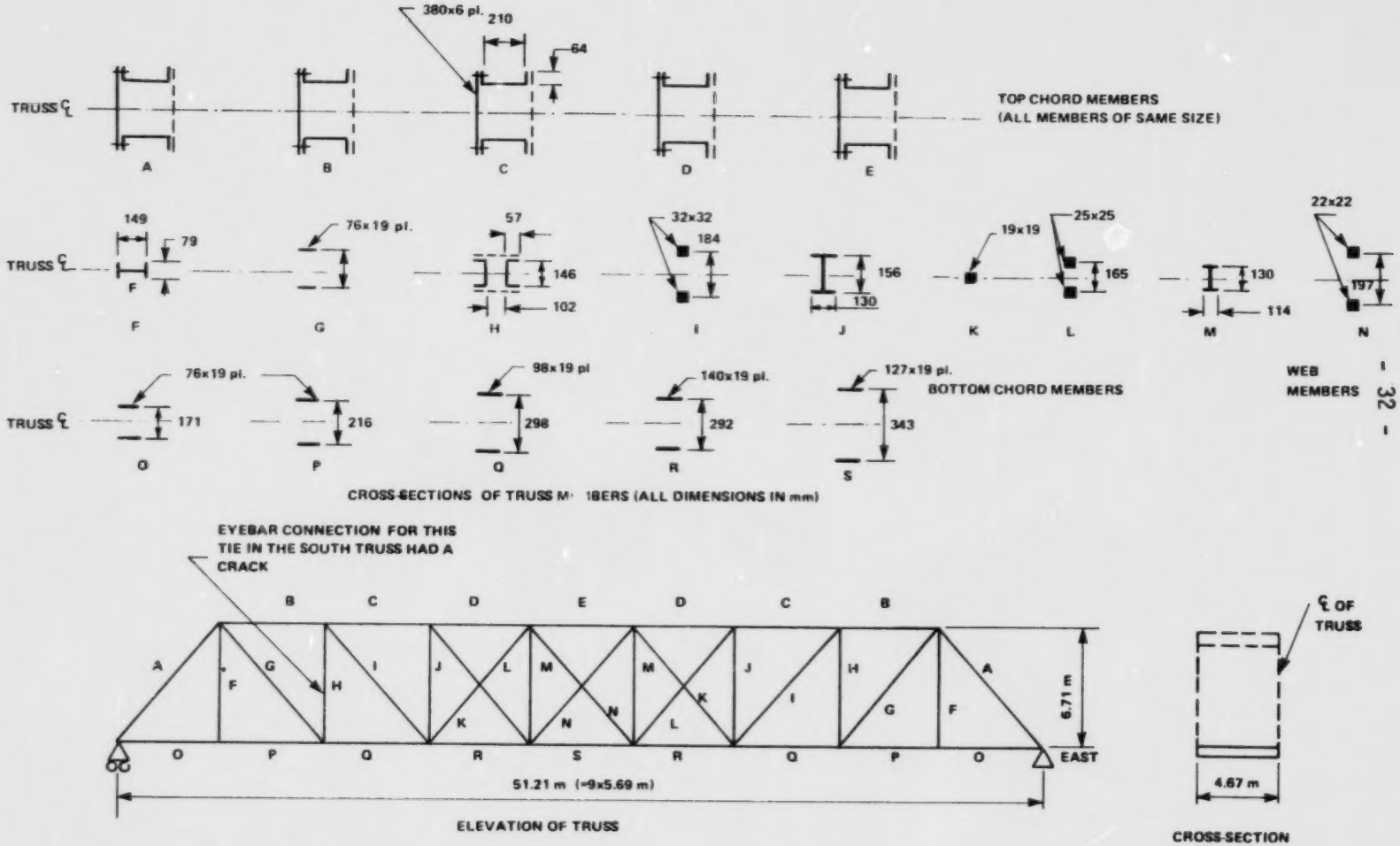
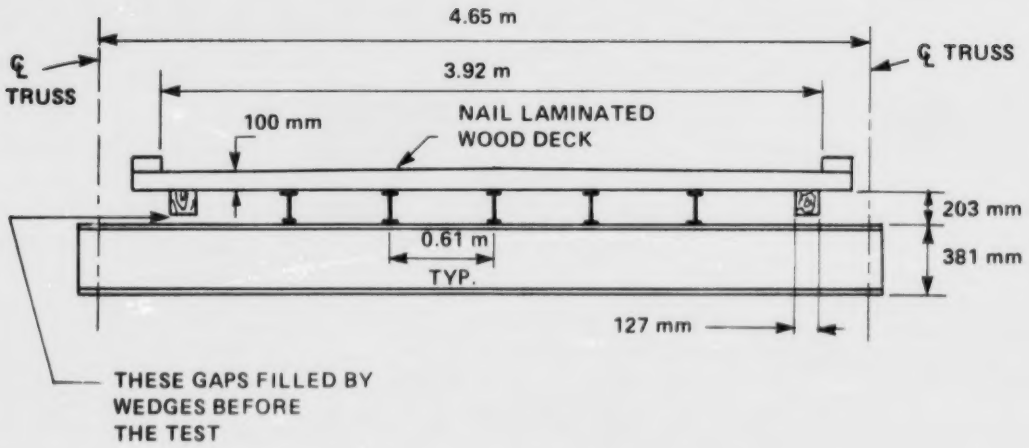


Figure 1.2/ Details of a Truss of The Finney Bridge



NOTE: STRINGERS ARE CONTINUOUS OVER TWO SPANS

Figure 1.3(a)/ Cross-Section of the Floor System

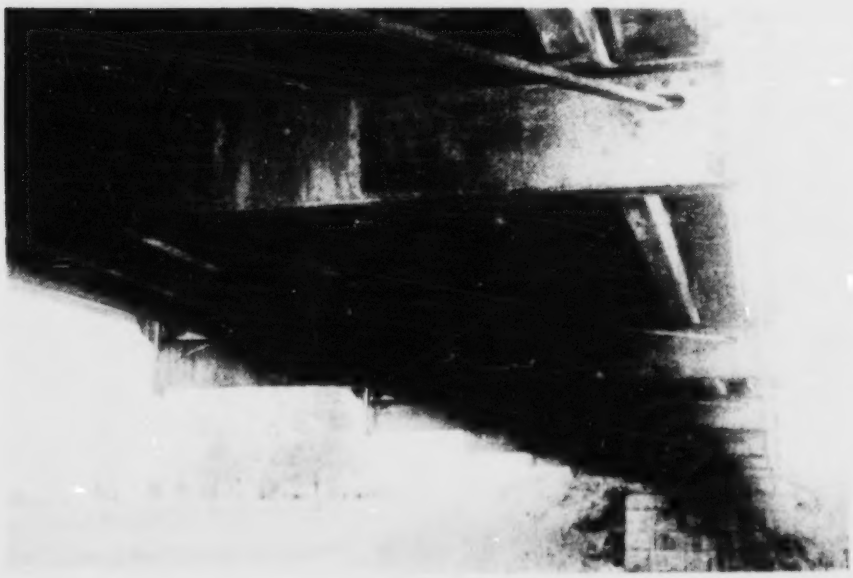
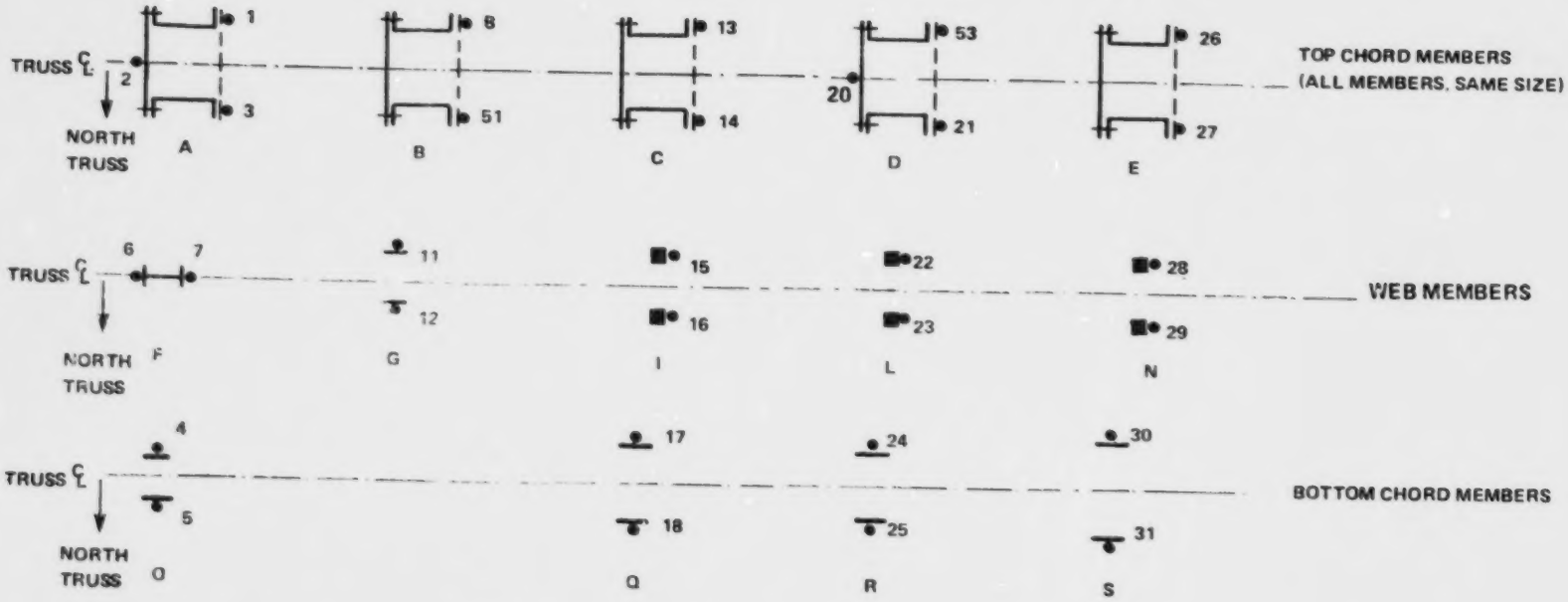
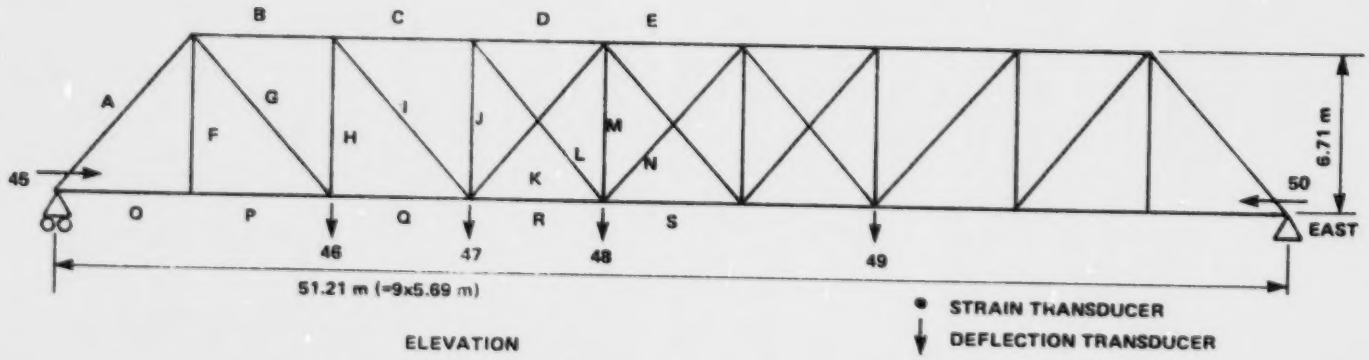


Figure 1.3(b)/ View of the Underside of the Flooring System





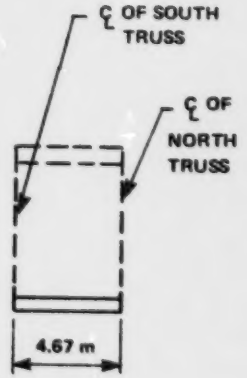
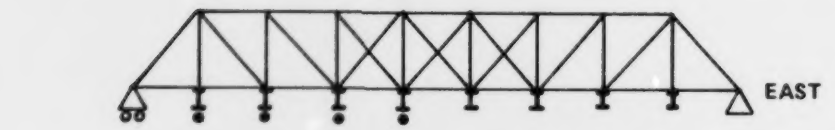


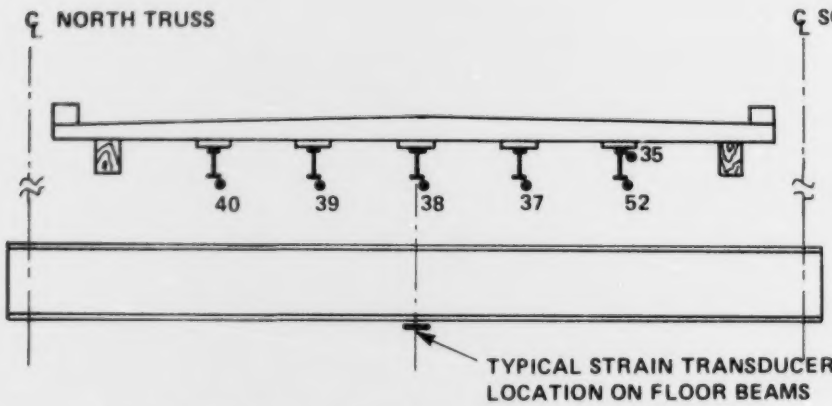
Figure 1.4/ Strain Transducer Locations, and Identifying Numbers, on the South Truss



41 42 43 44 IDENTIFICATION Nos. FOR STRAIN TRANSDUCERS ON FLOOR BEAMS

PANEL No. 1 2 3 4 5 6 7 8 9

ELEVATION



CROSS-SECTION (ONLY STRINGERS OF PANEL No. 2 INSTRUMENTED)

Figure 1.5/ Strain Transducer Locations on the Flooring System

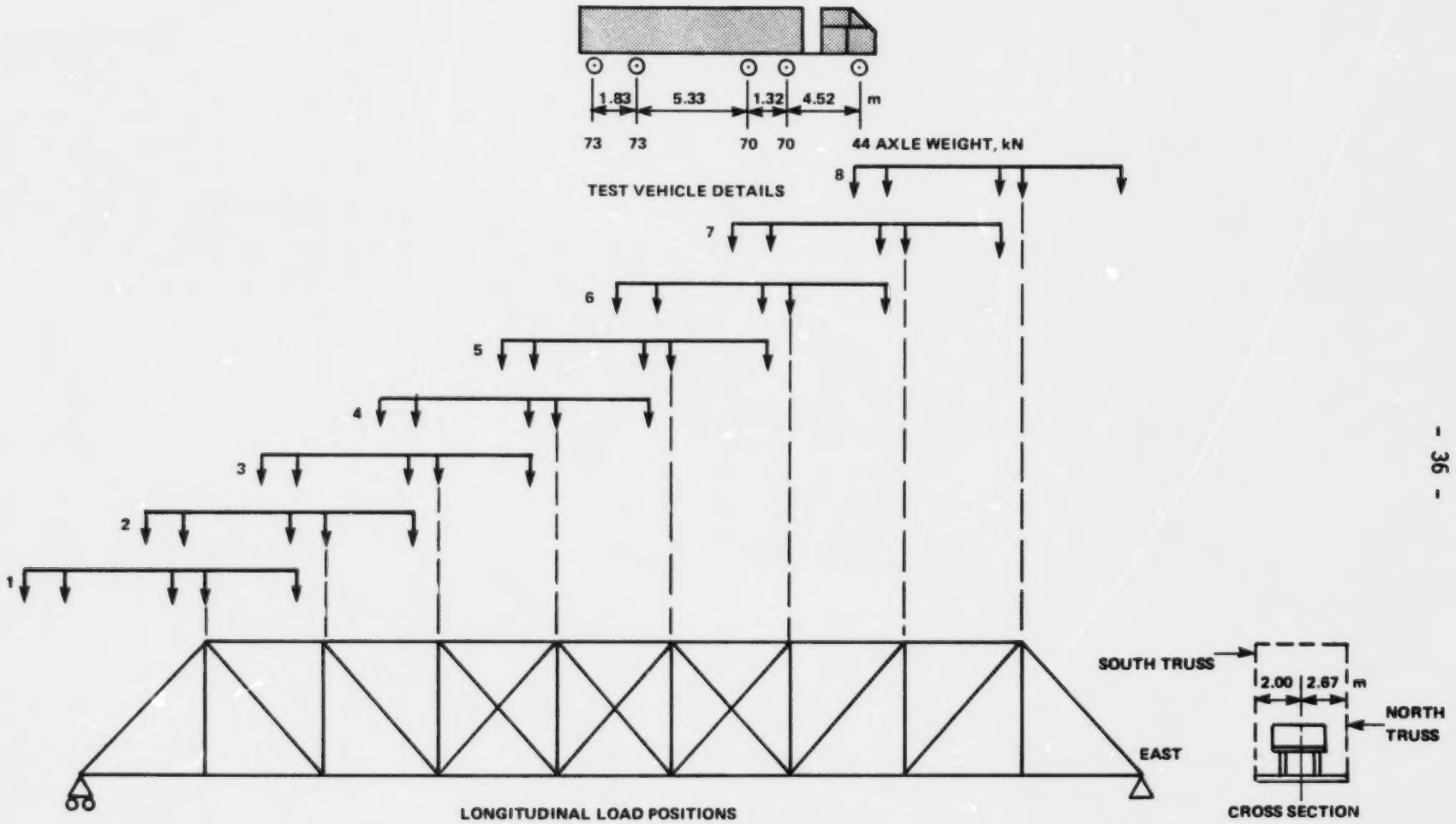


Figure 1.6/ Load Positions

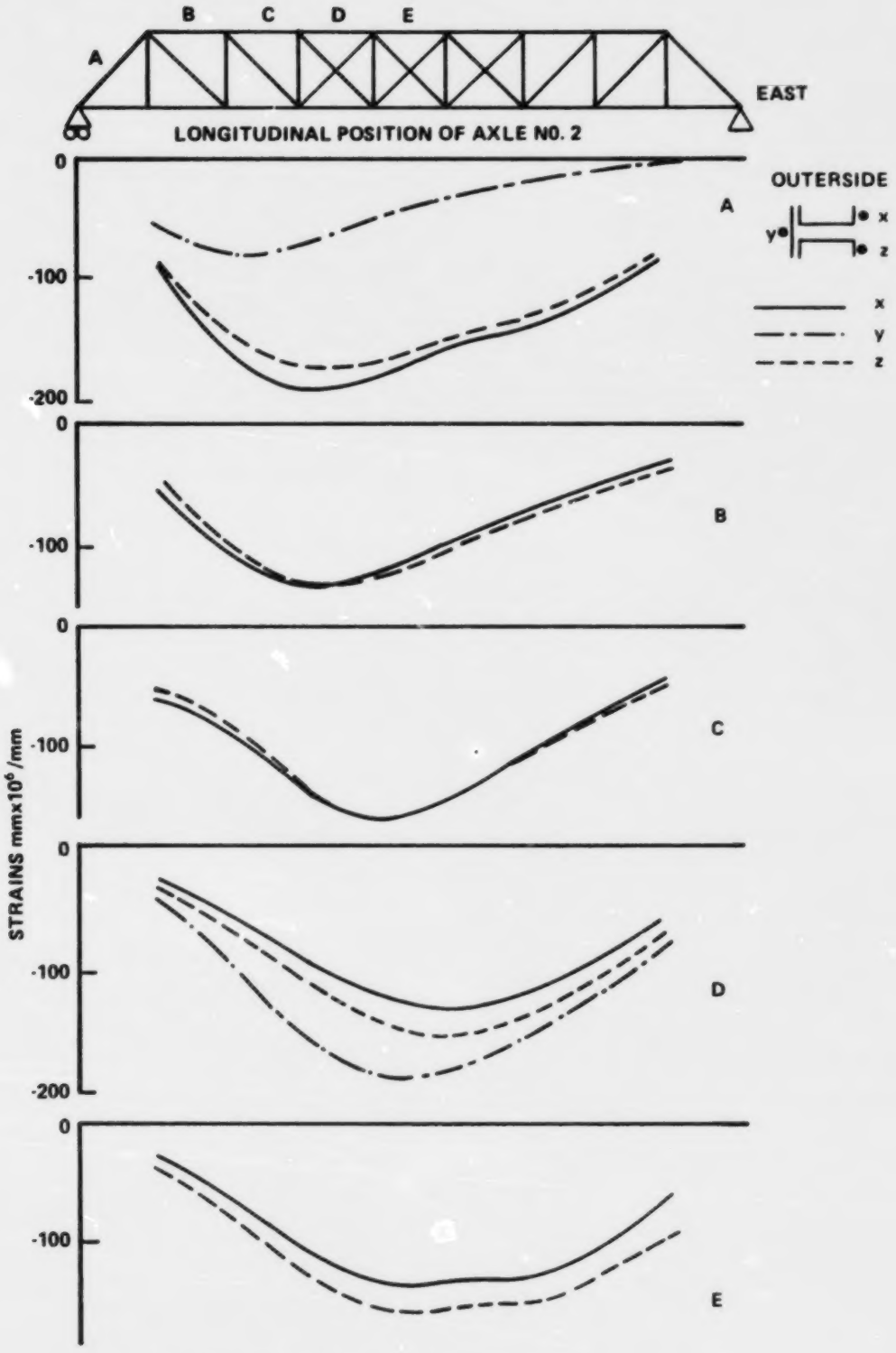


Figure 1.7/ Top Chord Strains

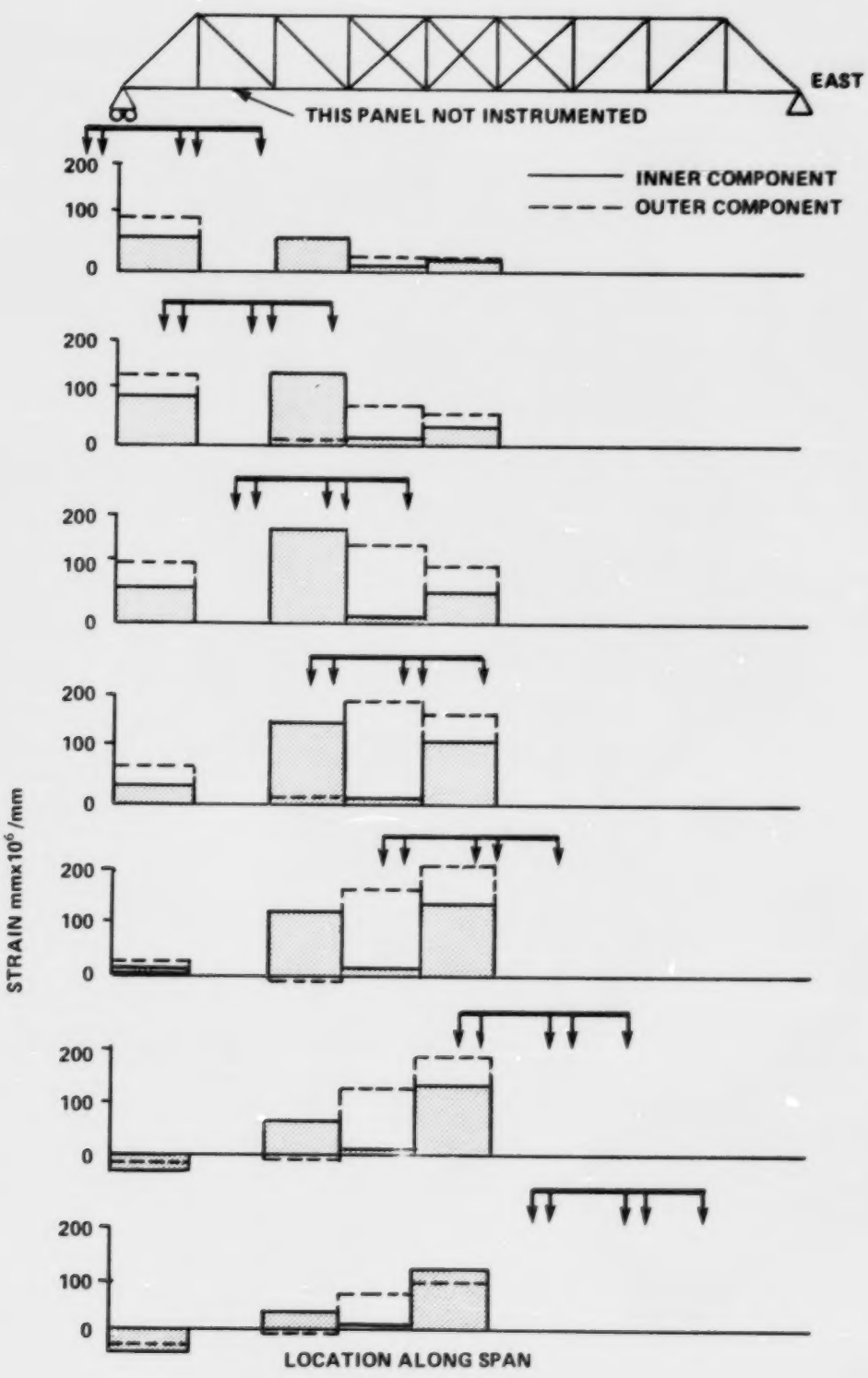


Figure 1.8/ Strains Along the Bottom Chord due to Various Load Positions

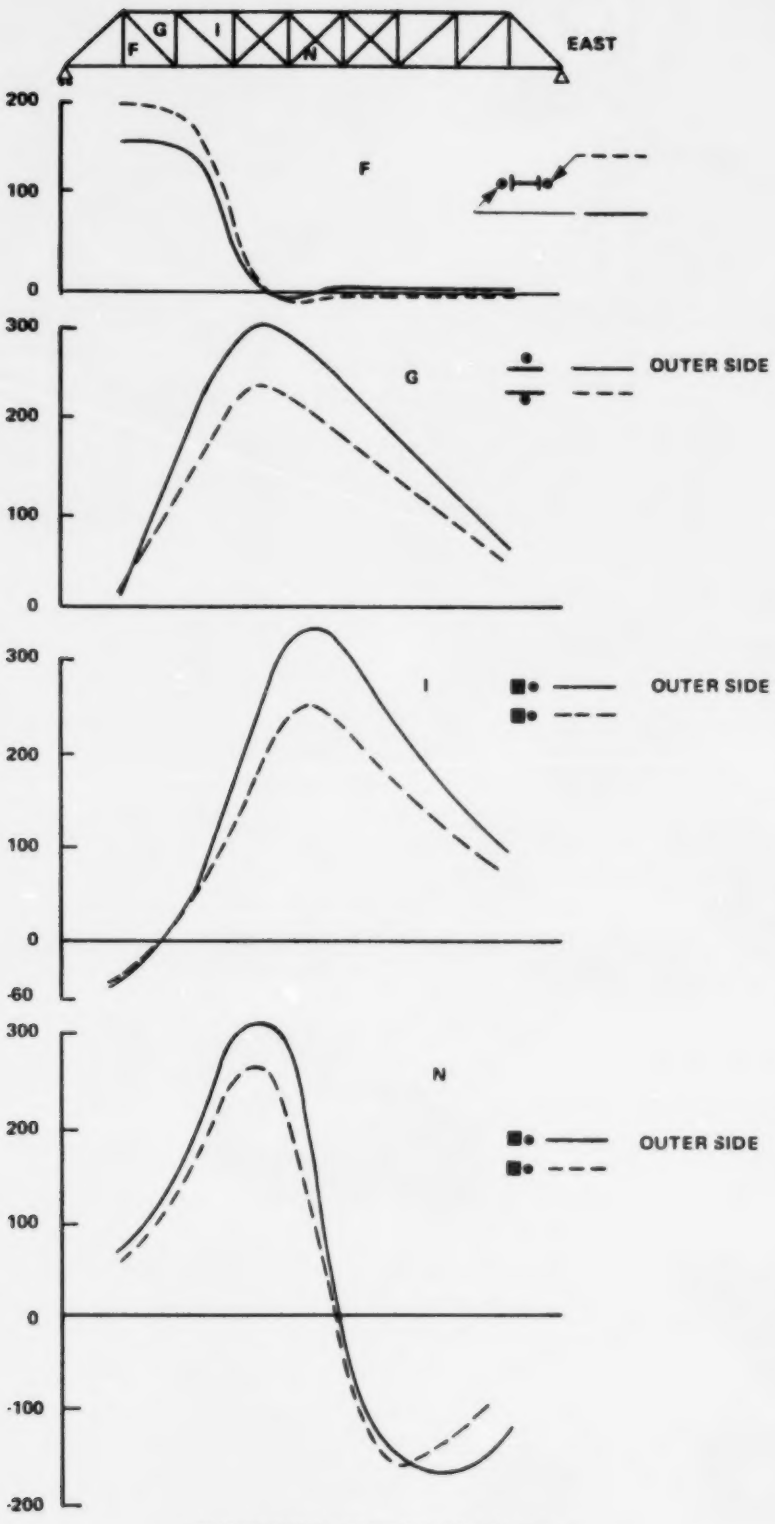


Figure 1.9/ Strain in Web Members

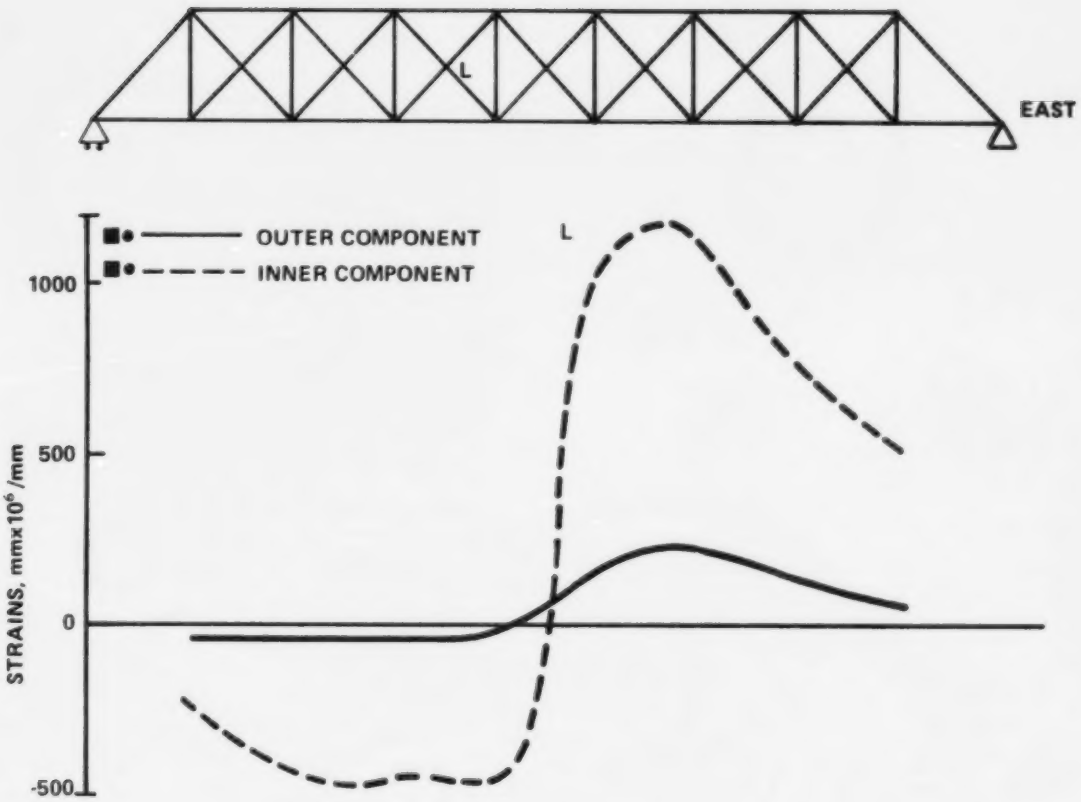


Figure 1.10/ Strains in an Inclined Member

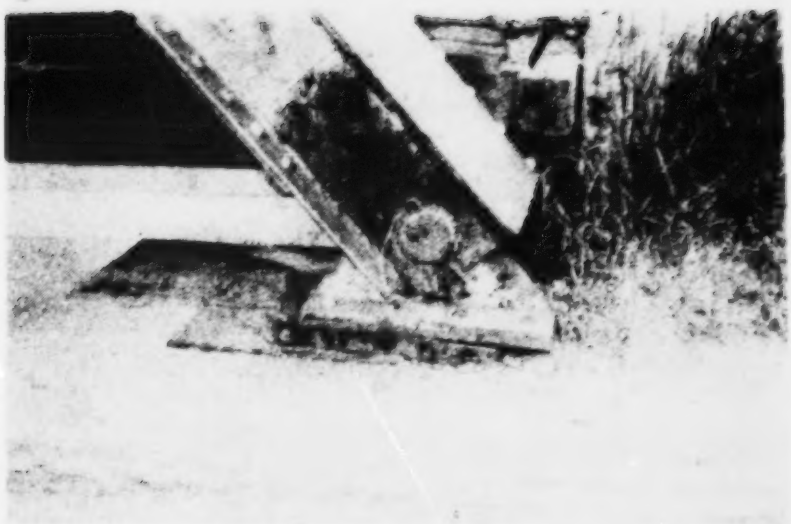


Figure 1.11/ A View of the Expansion Bearing

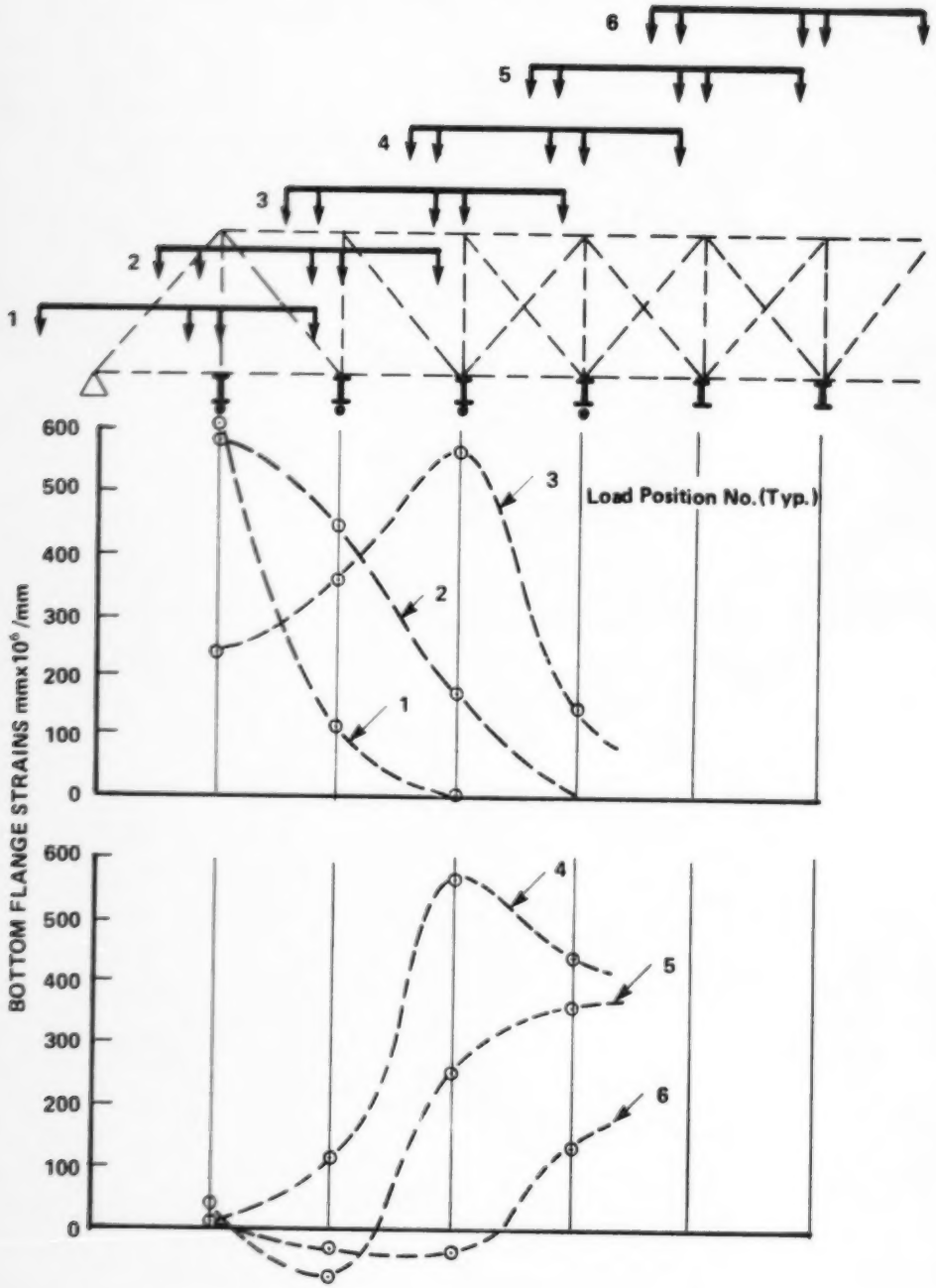


Figure 1.12/ Strains at Mid-Spans of Floor Beams

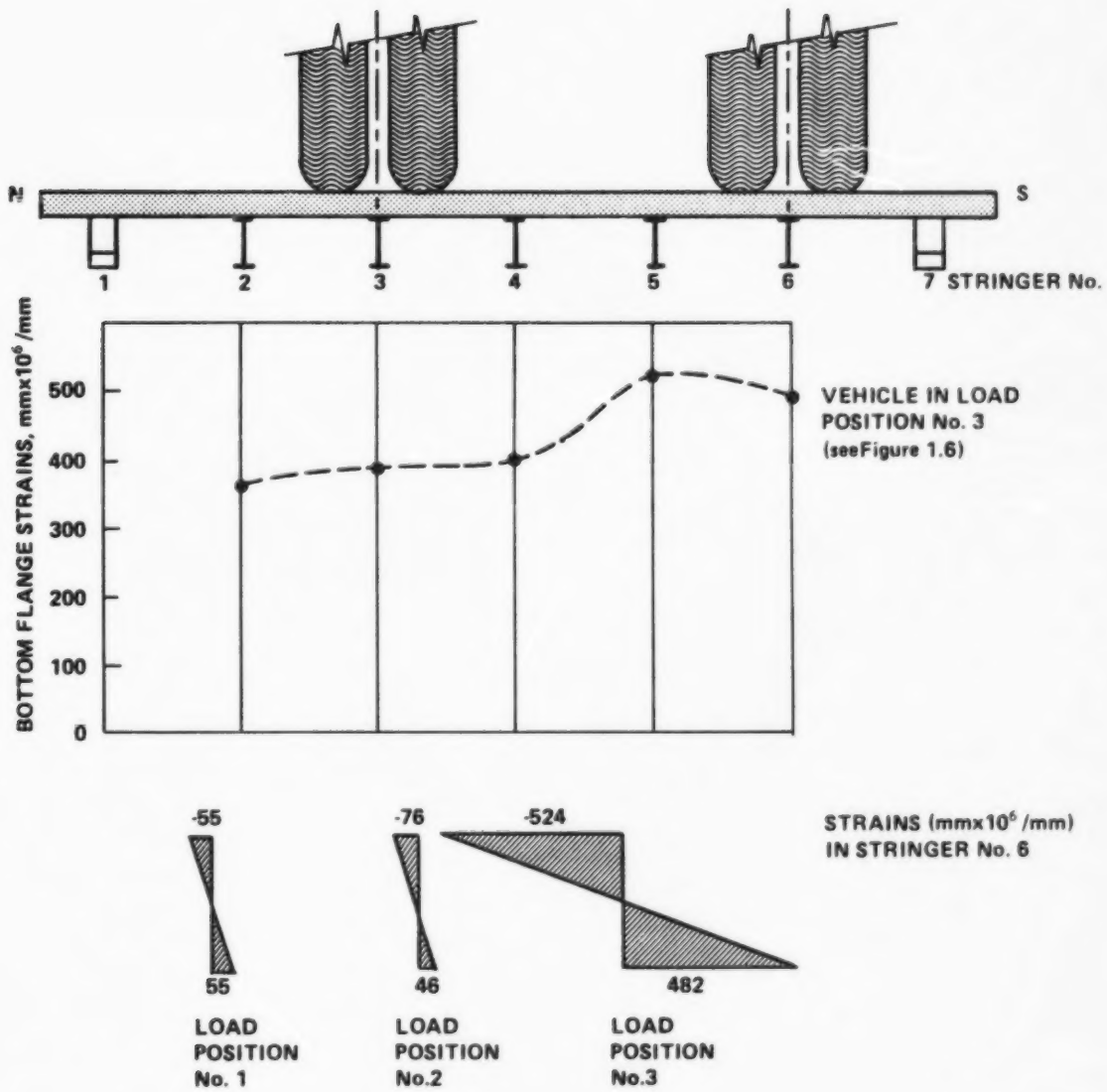


Figure 1.13/ Mid-Span Strains in Stringers of Second Panel from West Support

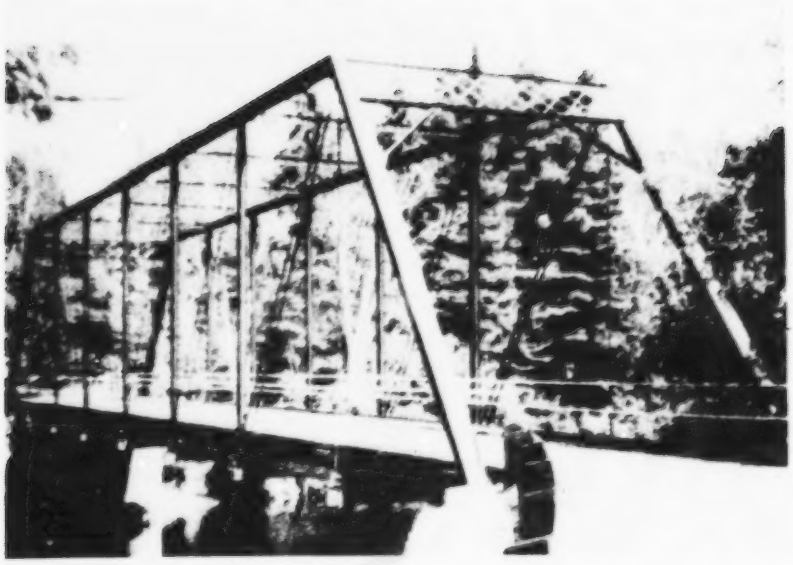
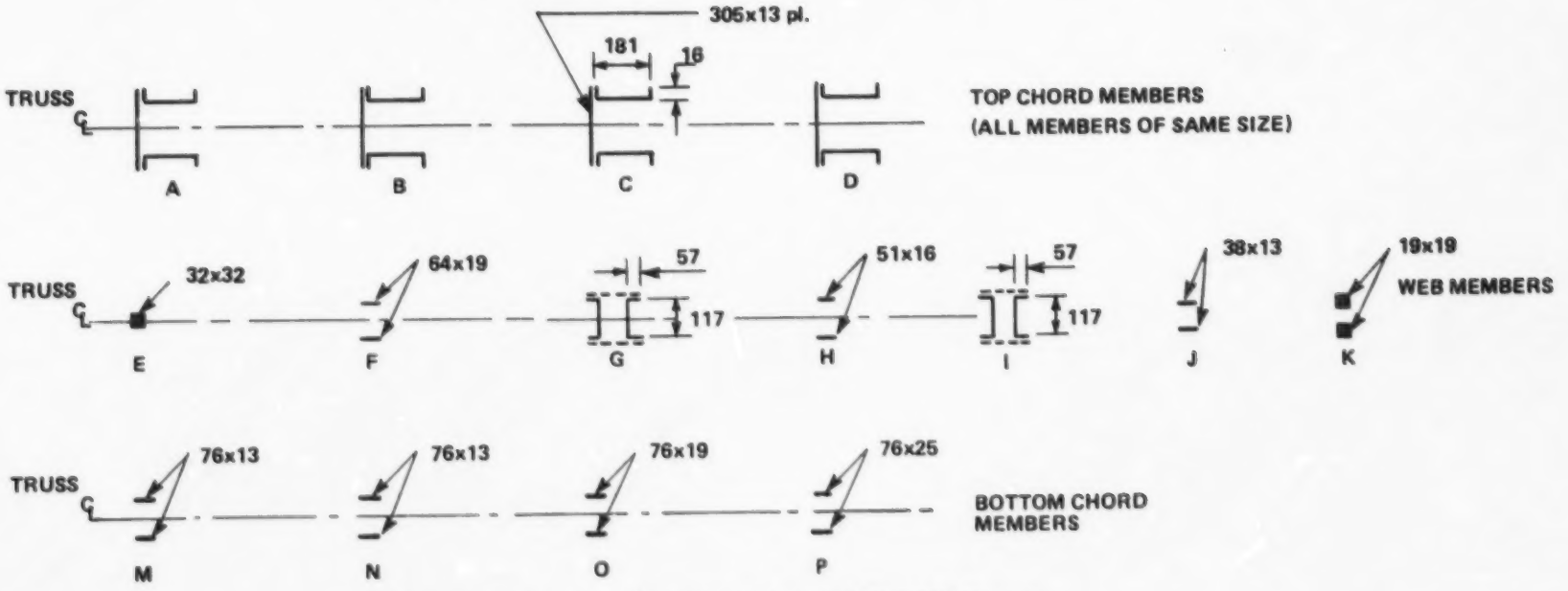


Figure 2.1/ A View of the Stephenson Townline Bridge



CROSS-SECTIONS OF TRUSS MEMBERS (ALL DIMENSIONS IN mm)

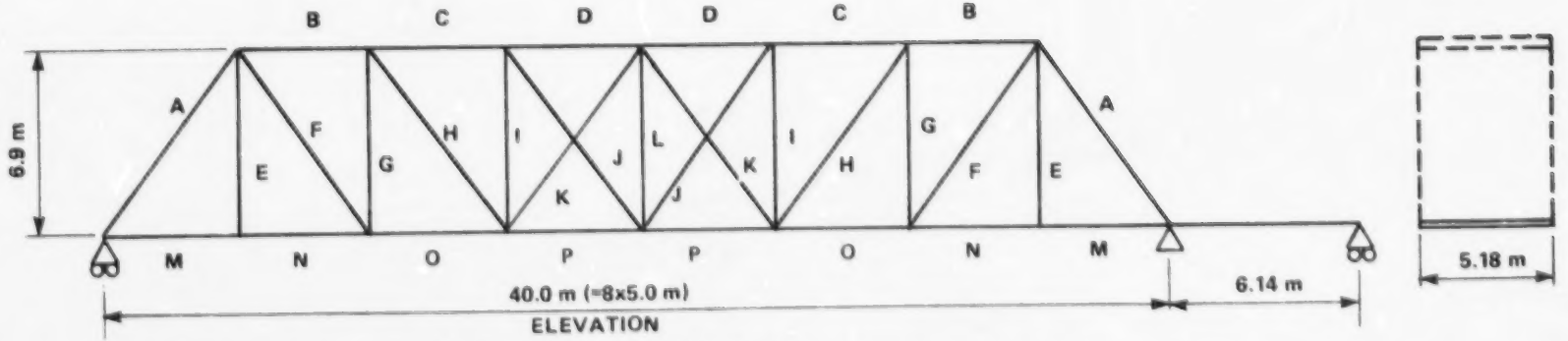


Figure 2.2/ Details of a Truss of the Stephenson Townline Bridge

Figure 2.3(a)/ A View of the Buckled Bottom Chord of the North Truss

Figure 2.3(b)/ A View of the Bottom Chord of the South Truss

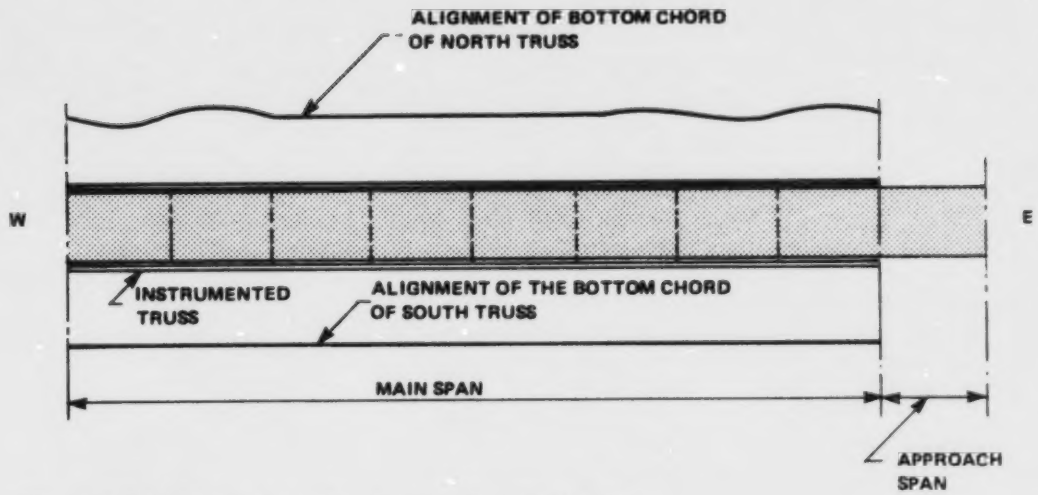


Figure 2.3 (c)/ Plan of the Stephenson Townline Bridge

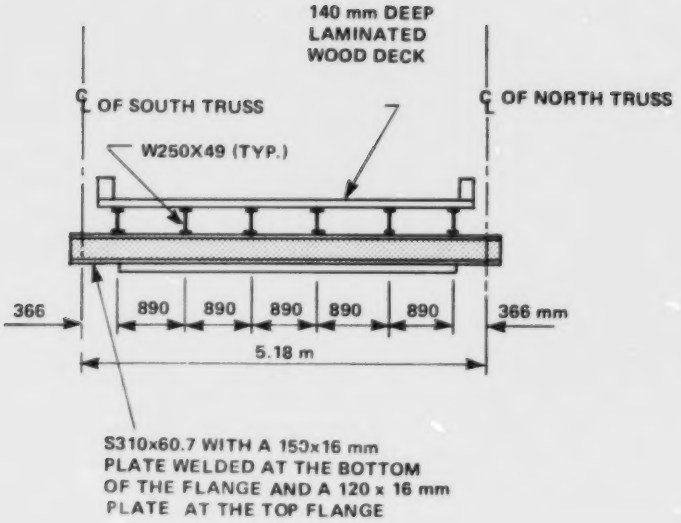


Figure 2.4/ Cross-Section of the Floor System

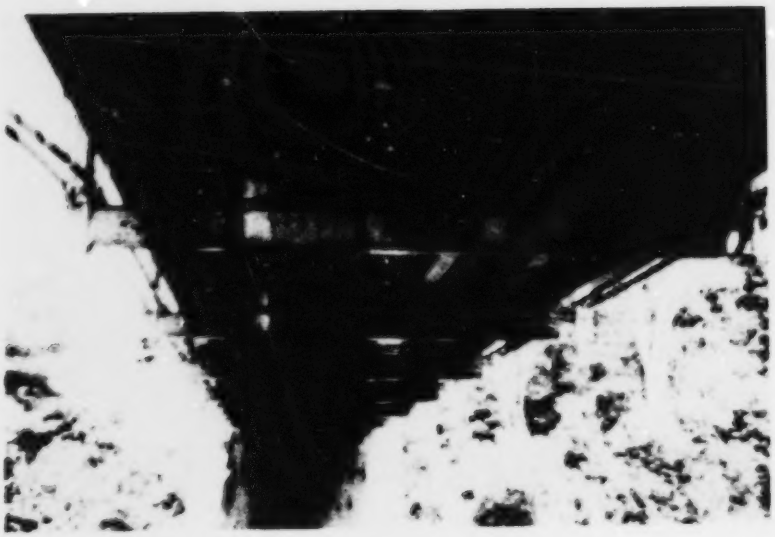
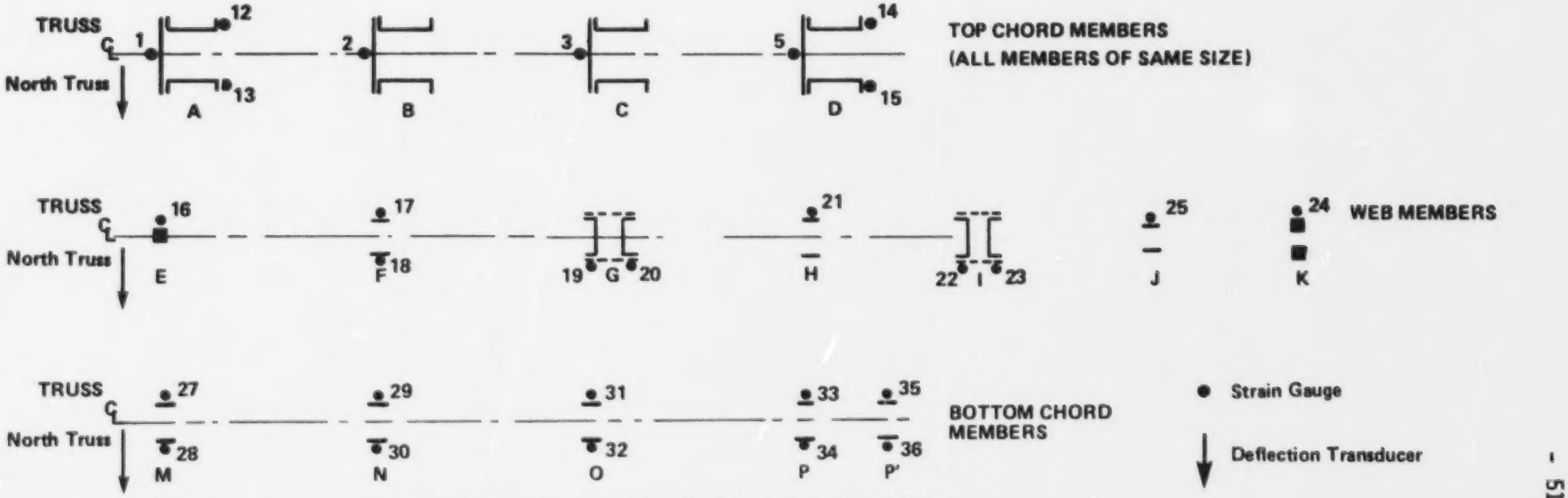


Figure 2.5/ A View of the Underside of the Floor System

Figure 2.6/ The Ti[illegible]-Level Posting Sign



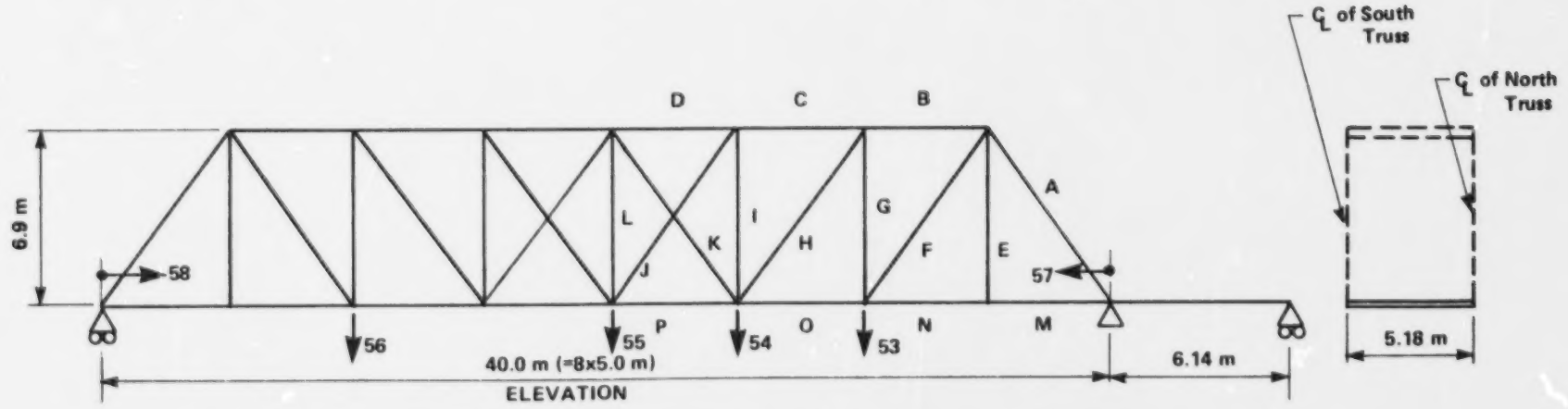
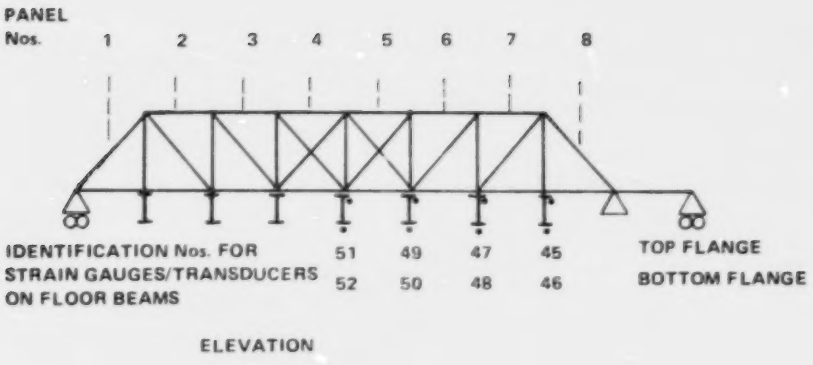


Figure 2.7/ Location of Instruments and Their Identifying Numbers on The South Truss



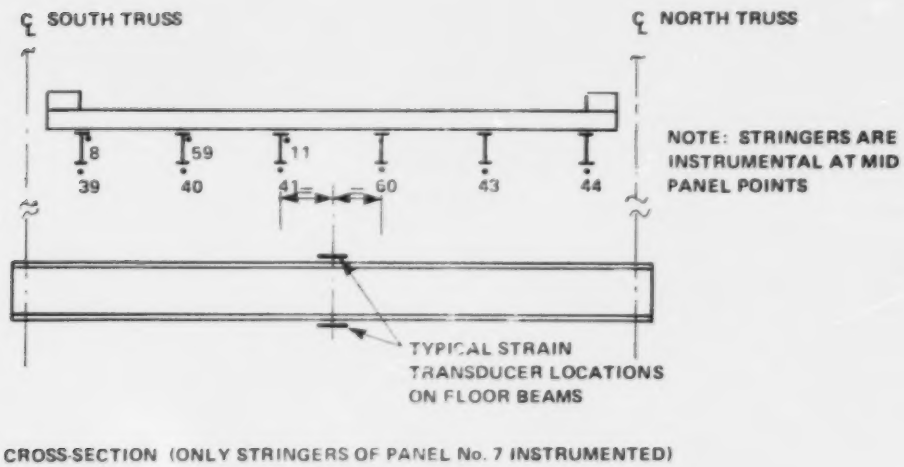


Figure 2.8/ Strain Gauge/Transducer Locations on the Flooring System

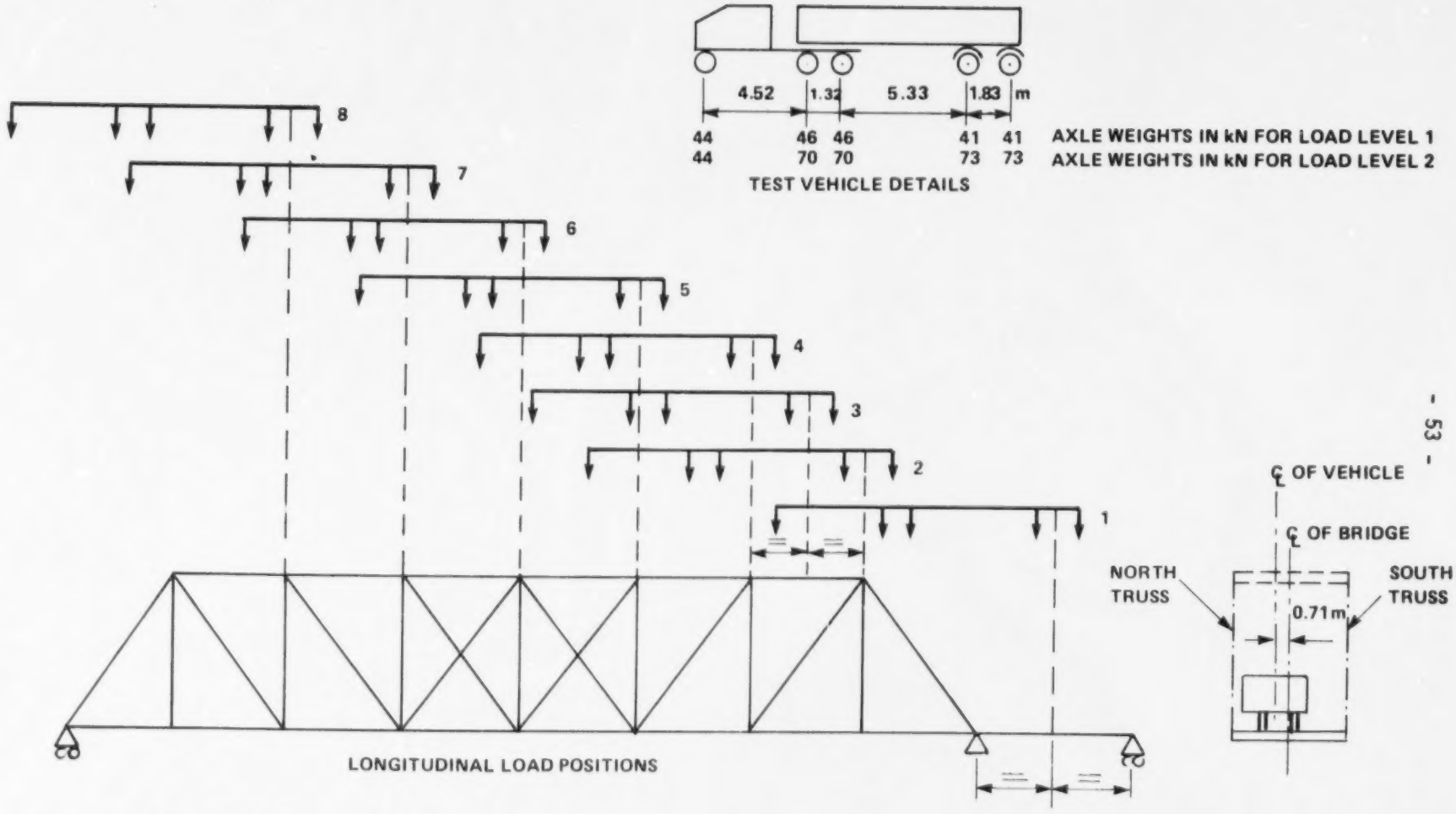


Figure 2.9/ Load Positions

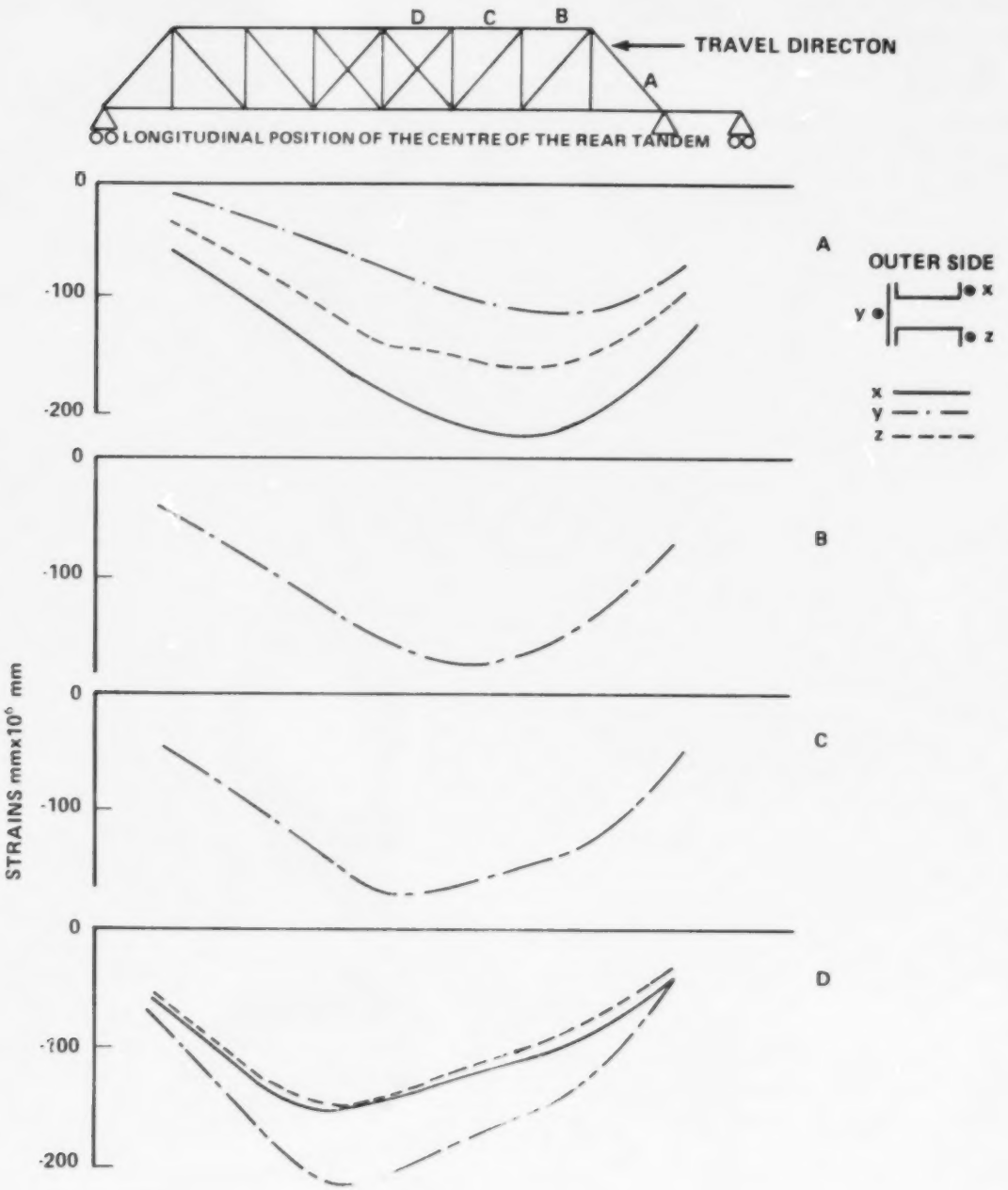


Figure 2.10/ Top Chord Strains in the Stephenson Townline Bridge

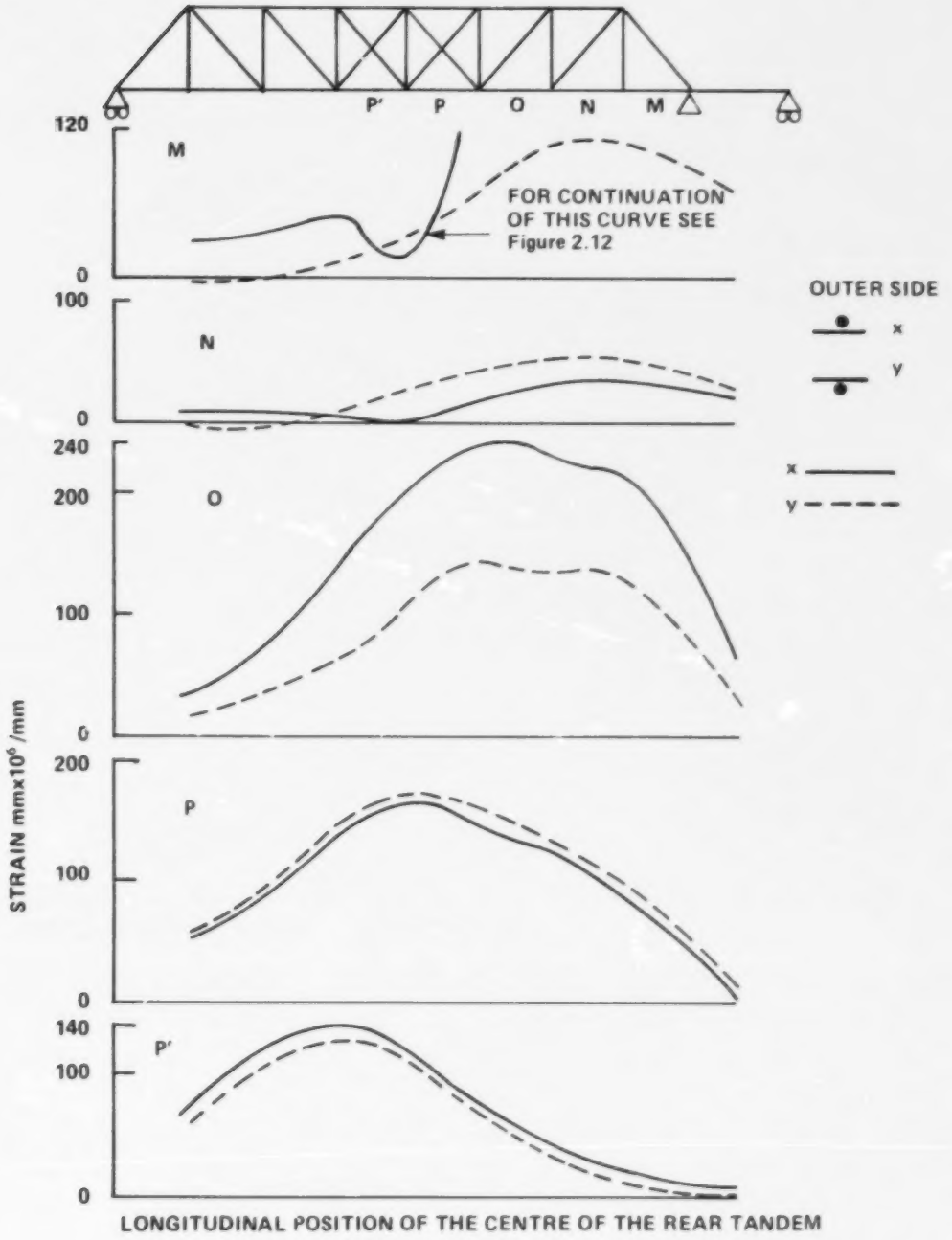


Figure 2.11/ Bottom Chord Strains Due to Load Level 2

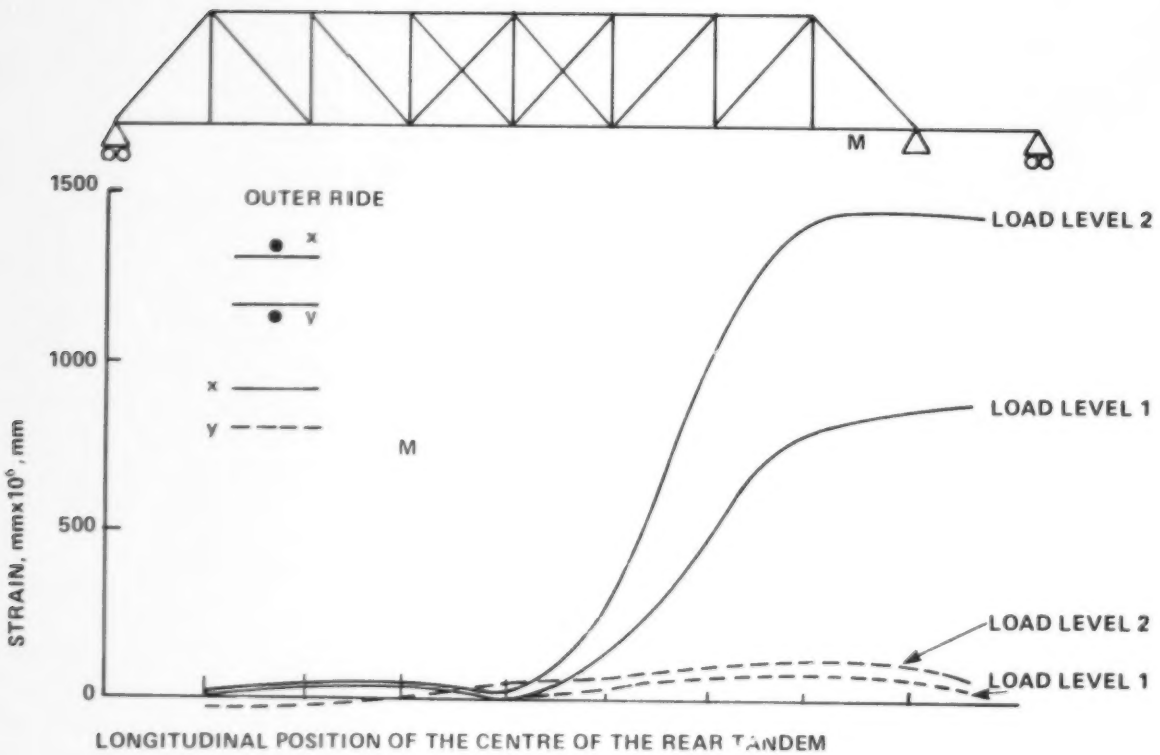


Figure 2.12/ Bottom Chord Strains in one Panel Due to Load Levels 1 and 2.

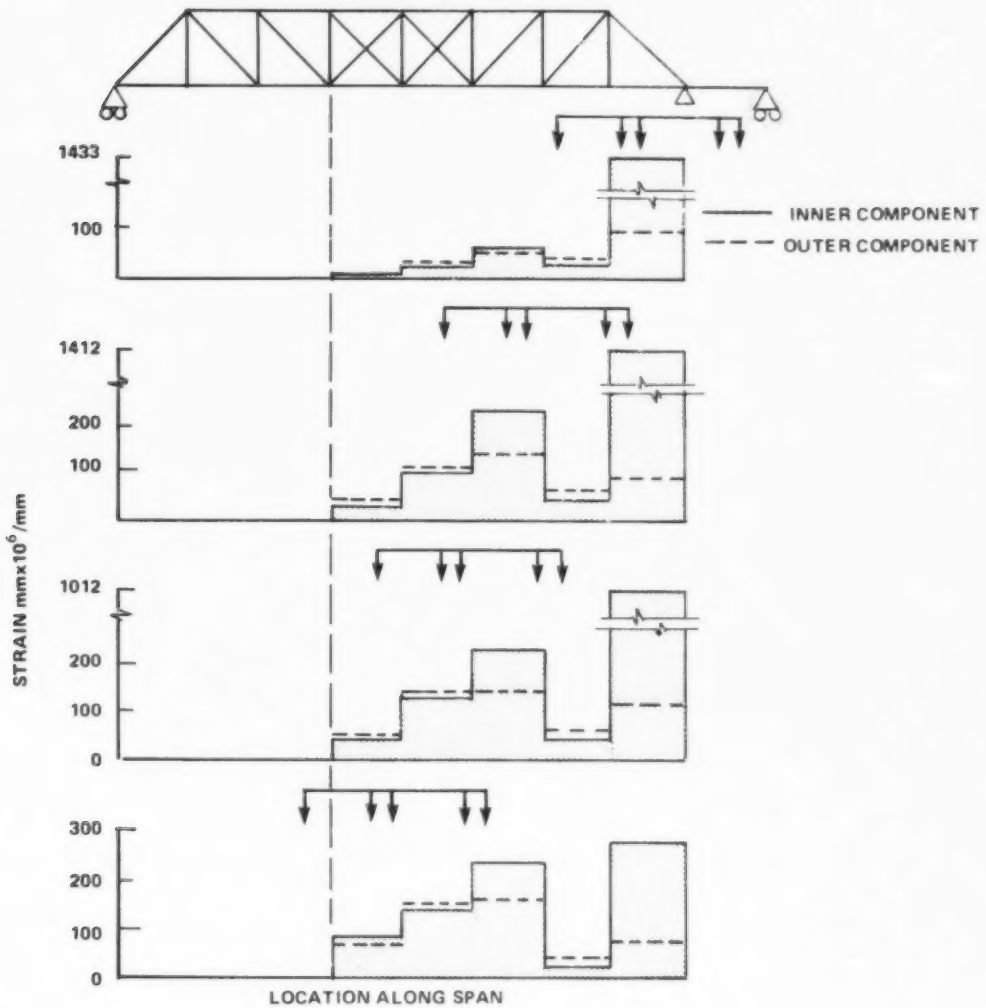


Figure 2.13/ Strains along the Bottom Chord due to Various Load Positions

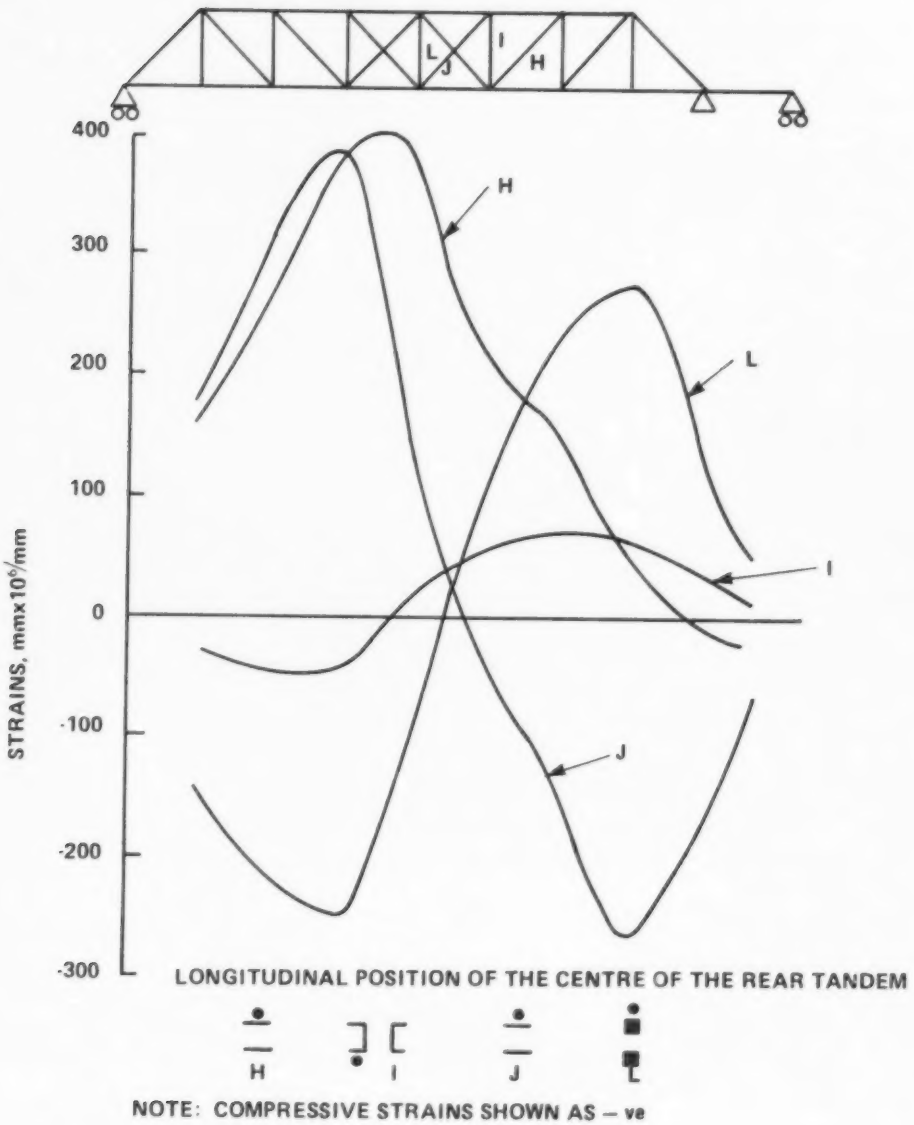
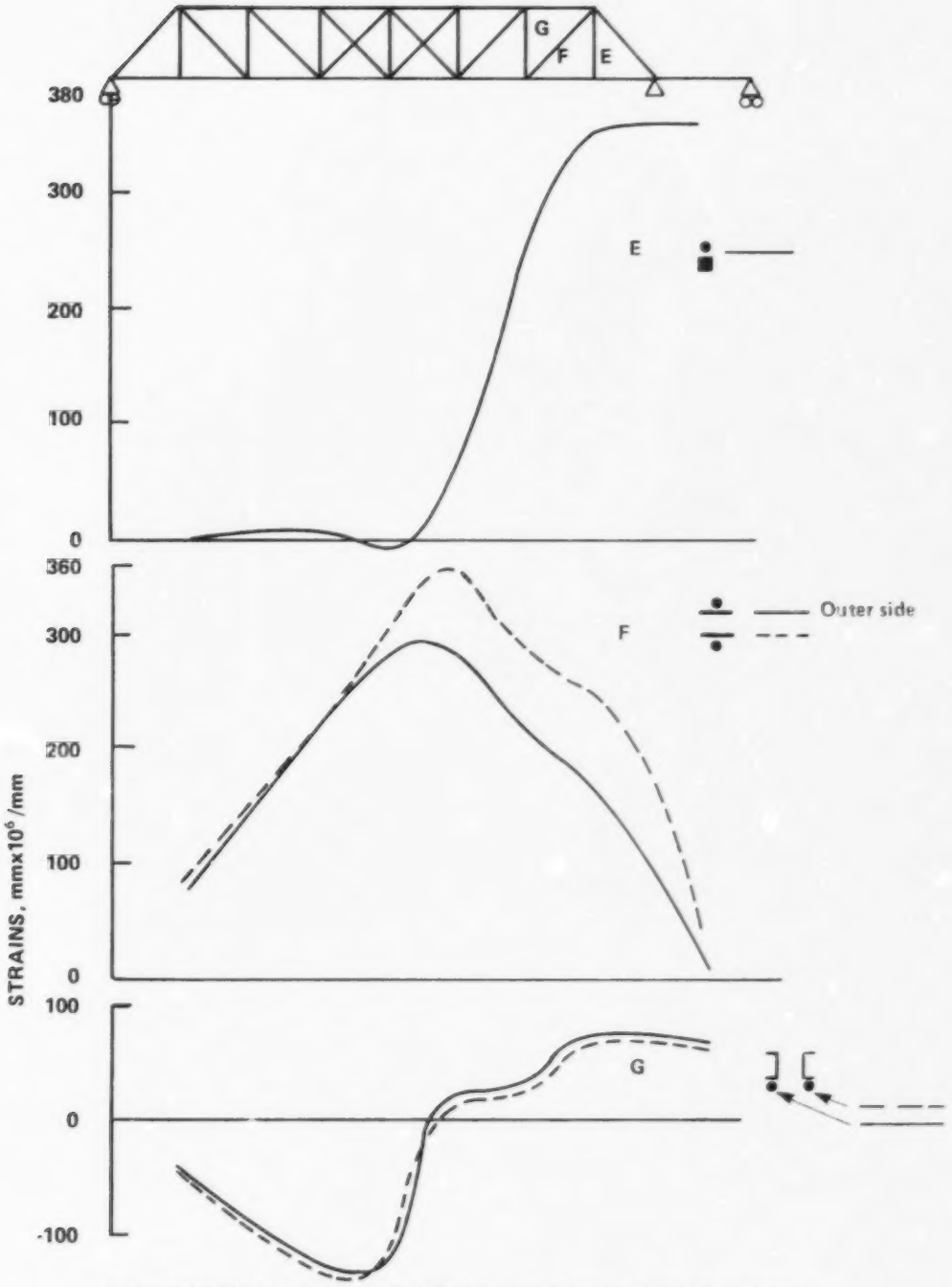


Figure 2.14/ Strains in Web Members



NOTE: COMPRESSIVE STRAINS SHOWN AS – ve

Figure 2.15/ Strains in Web Members

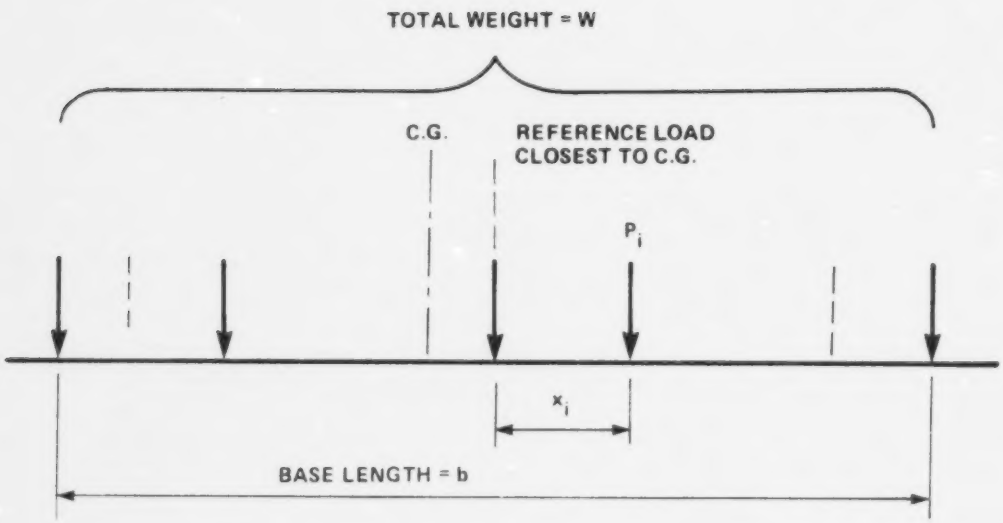


Figure 3.1/ Notation for Point Loads and their Spacings

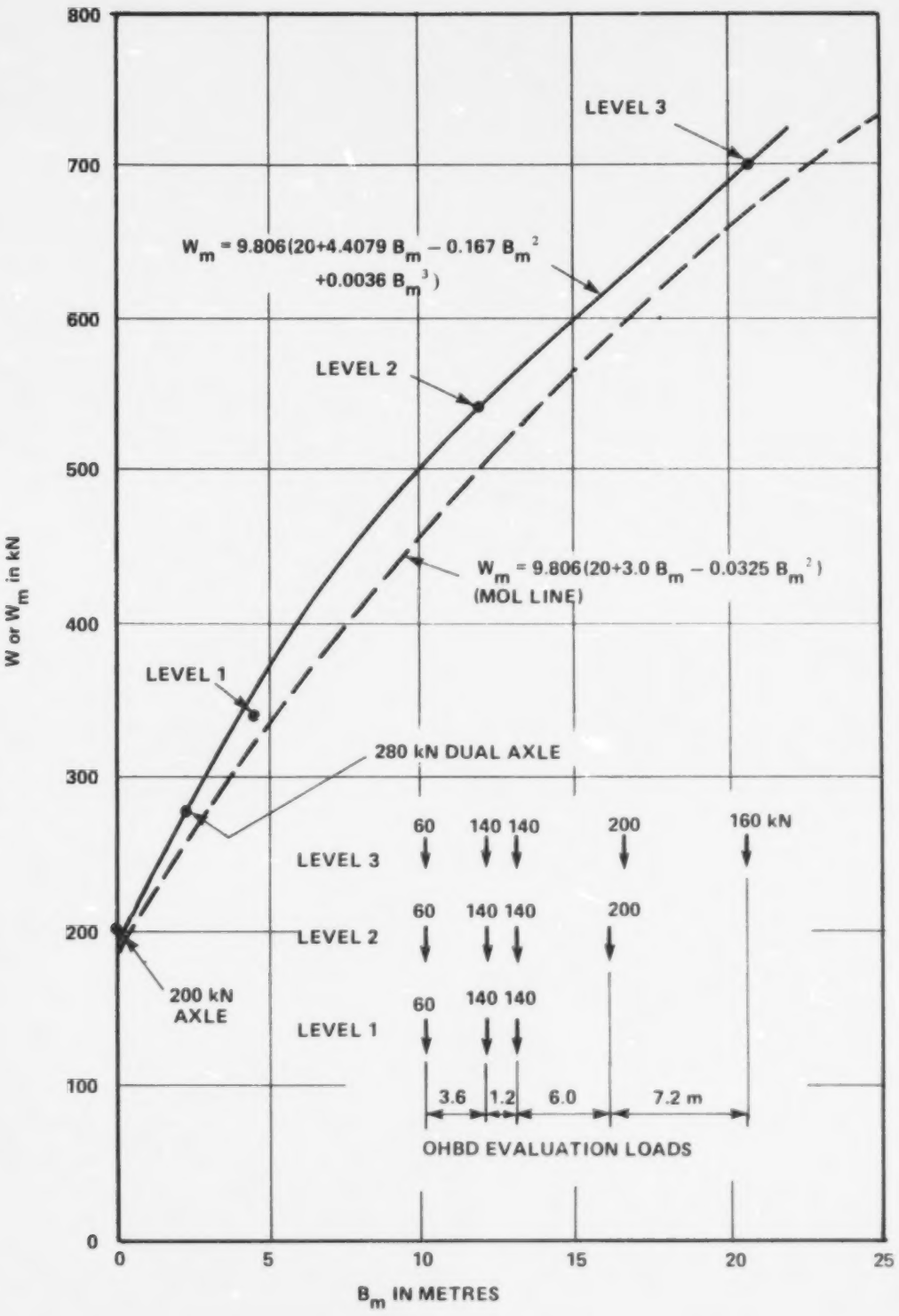


Figure 3.2/ Representation of a Design Truck by a Continuous Function